

A COMPREHENSIVE REVIEW ON ROCK BURST

by

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(1998)

Submitted to the
Department of Civil and Environmental Engineering
in partial fulfillment of the requirements for the degree of
Master of Science in Civil and Environmental Engineering
at the

MASSACHUSETTS INSTITUTE OF TECHNOLOGY

August 2000

[September 2000]

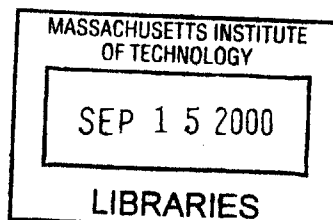
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BARKER

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ABSTRACT

The term “rock burst” is commonly used to describe a wide range of rock failures, which occur in tunnels, shafts, caverns and mines. Rock burst is a sudden form of rock failure characterized by the breaking up and expulsion of rock from its surroundings accompanied by a violent release of energy. Rock bursts have been encountered frequently in mines and on few occasions in civil engineering tunnels. Rock bursts in tunnels represent a significant threat to the safety of tunnel workers and equipment, as well as a significant factor with respect to performance of tunneling contracts. The increasing amount of civil engineering tunnels built in difficult geologic conditions and under high overburden makes an understanding of rock bursts necessary.

A summary of available research on bursts, case studies, used and suggested support systems, design methods, prevention and prediction procedures are presented. Several different mechanisms of rock bursting are described, and a clear distinction between source and damage mechanism is drawn. Criteria for identifying burst prone ground, conditions and thoughts concerning support of tunnels are presented. Practical implications from experiences in mining for permanent civil engineering structures are pointed out.

Support systems are evaluated with respect of their ability to withstand rock bursts. A rationale for designing support systems is given. It involves the identification of the likely modes of failure in the tunnel and the determination of support survival limits. Also, methods of rock burst prediction used in mining and methods that prevent rock bursts by eliminating the trigger mechanism are presented.

Thesis Supervisor: Prof. Herbert H. Einstein

Title: A Comprehensive Review on Rock Burst

Acknowledgements

I would like to express appreciation to the following individuals for helping to make this thesis possible:

Prof. Herbert H. Einstein, my thesis supervisor for his teaching, assisting and guiding me throughout all phases of my research. His insight lead me through this new and challenging experience. I will always be grateful for his high level of support with all the problems we encountered and for his positive and cheerful attitude that helped carry me through the tough times.

The faculty of the Mechanics and Material Group at M.I.T.

My friends and colleagues at M.I.T. who helped make this a memorable experience. Alex Liakos, Jorge Gonzalez, Federico Pinto, Catalina Marulanda, Christoph Haas, Kurt Sjoblom, Sanjay Pahuja, Karim Karam, Yun Sun Kim, Greg DaRe, Hannes Lager, Attasit “Pong” Korchaiyapruk, Suchatvee “Vince” Suwansawat, Maria Santagata, Dominic Assimaki, Babar Bhatti, Gouping Zhang, Laurent Levy, Cathy Castenson, Kortney Adams, Kartal Toker, Vladimir Ivanov, Alice Kalemkarian and “The Greeks”. Their friendship is a treasure that I will carry forever.

All dirtballs, labrats and geotechs that have come and gone.

Last but not least, I would like to thank my family, who has always supported, loved and believed in me throughout every step of my life.

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Chapter 1 Introduction

1.1 Background and Problem Statement

Rock bursts are explosive failures of rock mass around an underground opening, which occur when very high stress concentrations are induced around underground openings (Hoek, 2000). Rock burst problems are particularly acute in deep level mining in hard brittle rock. However rock bursts have been observed in a number of tunnels, shafts and other permanent underground structures in hard brittle rock. The latter will be the main focus of this thesis. Rock bursts in tunnels represent a significant threat to the safety of tunnel workers and equipment, as well as a significant factor with respect to performance of tunneling contracts. As depths and locations have become increasingly challenging more cases of rock bursting in tunnels have been reported. Chapter 2 of this thesis provides more details on mechanisms, causes and impact.

Rock bursts have occurred in many countries. However, most research was carried out in a fairly isolated manner. Most studies of rock burst events were done in individual localities without relating them to other regions. Similarly, significant related research in rock mechanics carried out over the last 25 years in countries like Russia, Japan and China was almost unnoticed and difficult to access for the English-speaking world. For this reason the cases and sources in this thesis are somewhat focused geographically on the U.S., Canada, Europe and South Africa unless a project with international participation is involved.

Some of the mines where rock bursts have occurred in, are the deep level gold mines in the Witwatersrand area in South Africa, the Kolar gold mines in India, the El Teniente copper mine in Chile, the Canadian nickel mines in Sudbury, the Idaho Coeur d'Alene area mines in the US and the gold mines in the Kalgoorlie area in Australia. Some engineering structures where rock bursts have occurred are underground openings for hydro-electrical power plants in China and Norway, Japanese Tunnels and Norwegian tunnels.

Because rock bursts, are much more common in mines than in tunnels, shafts and caverns most available research is in mining engineering rather than civil engineering. Even though publications show that there are differences in the conceptual understanding of rock bursts that seem to root from the background of the authors (Myrvang & Grimstad, 1983 vs. Ortlepp, 1979), rock bursts always involve the violent ejection of rock from the surface of the tunnel whether encountered in mining or civil engineering. The practical implementation however can be significantly different depending on the purpose of the structure. For both mines and civil engineering underground openings the main problem rock bursts create is safety during underground work. Efforts have been made to find methods to predict rock bursts, to develop support systems that can resist rock bursts or appropriate countermeasures that will prevent rock bursts.

1.2 Research Objectives

The main focus of this thesis is on permanent engineering structures like tunnels, shafts and caverns. The thesis provides an overview of research carried out on rock bursts in these structures as well as relevant rock burst research in the mining area. It demonstrates in case studies that rock bursts have significant impact on the overall project cost and time and are not only a structural problem. The summary shows that conclusions from the available research and the practical countermeasures against rock burst are diverse. It can be seen that there might not be one best approach how to deal with rock burst problems. Therefore this thesis presents available options and illustrates on the basis of case studies how these were applied.

Rock burst events are relatively concentrated in a few regions in the world. From the cases presented one gets an impression in which regions and conditions rock bursts are most likely to occur. Methods on how to predict rock bursts are presented. Exact temporal and spatial prediction of rock bursts will not be possible, but the cases and methods presented will help to evaluate potential rock burst risk.

1.3 Organization

Chapter 2 describes general characteristics of rock bursts. The difference in source and damage mechanisms is explained. Causes and factors influencing rock bursts are discussed.

Chapter 3 presents case studies on rock bursts in civil engineering projects in Norway, China, Japan and the United States.

Chapter 4 presents observations from damage to tunnel support from rock bursts. Functions, components and desired characteristics of support systems for rock burst conditions are described. Support systems currently used in mines for rock burst conditions are presented. Design rationales, considerations and examples are introduced.

Chapter 5 presents methods for prediction and prevention of rock bursts.

Chapter 6 presents a summary of the thesis, conclusions from the research and recommendations for future research on this topic.

Literature

Hoek, E. (2000); Practical Rock Engineering 2000 edition, work in progress

Myrvang, A.; Grimstad, E. (1983); „Rockburst Problems in Norwegian Highway Tunnels-Recent Case Histories“; In Rockburst Prediction and Control; p.133-139; Institute of Mining and Metallurgy

Ortlepp, W.D. (1978); The Mechanism of a Rockburst”; In Proc. 19th U.S. Symposium on Rock Mechanics, May 1978, Stateline, Nevada; p.478-483; ISRM

Chapter 2 Characteristics of Rock Bursts

2.1 General Characteristics of Rock Bursts

2.1.1 Description

The term “rock burst” is commonly used to describe a wide range of rock failures, which occur in tunnels, shafts, caverns and mines. It is used to describe the expulsion of small rock fragments from the surface of mine pillars or the tunnel perimeter, which sometimes is referred to as “spitting rock”, as well as for the sudden collapse of a pillared mining area greater than hundred thousands of m². Some examples of underground openings damaged by rock bursts can be seen in Figure 2.1 to Figure 2.4. Blake (1972) describes rock burst as “a sudden form of rock failure characterized by the breaking up and expulsion of rock from its surroundings accompanied by a violent release of energy”. Hoek (2000) states that “rock bursts are explosive failures of rock mass around an underground opening, which occur when very high stress concentrations are induced around underground openings”. By adding the phase of mining or the cause to it, the term “rock burst” is sometimes used more restrictively. Terms such as “pillar bursts”, “pressure bursts”, “strain bursts”, “crush bursts”, “inherent bursts” and “induced bursts” and many more can be found in the literature.



Figure 2.1: Rock Burst Damage in a Gallery (U.S. Bureau of Mines, Spokane)



Figure 2.2: Rock Burst Damage in a Gallery (U.S. Bureau of Mines, Spokane)



Figure 2.3: Heavy Rock Burst Damage (U.S. Bureau of Mines, Spokane)

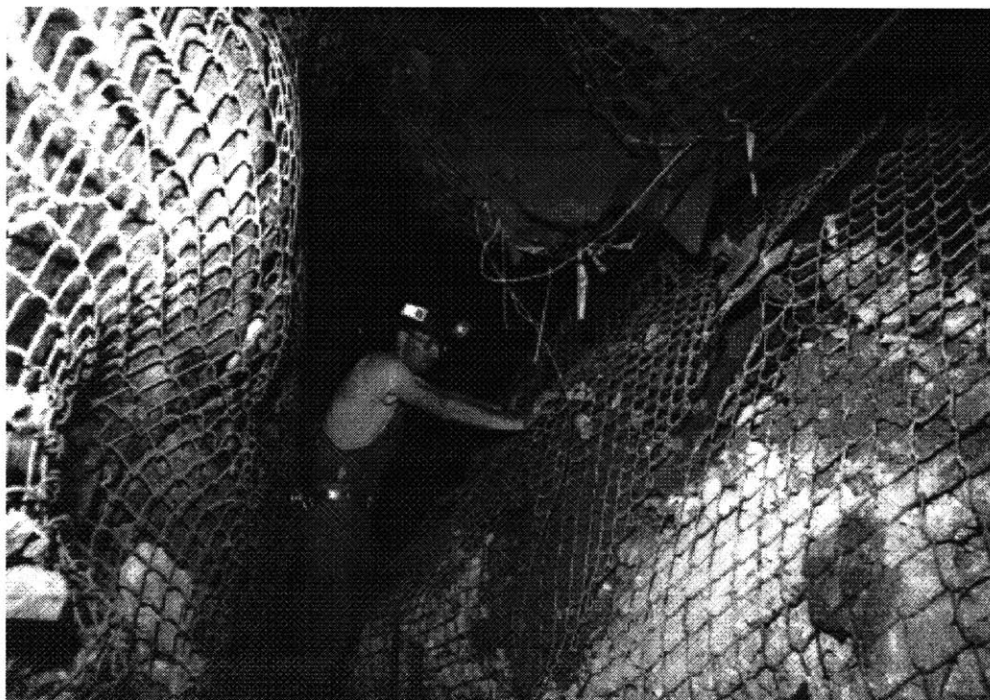


Figure 2.4: Heavy Rock Burst Damage (U.S. Bureau of Mines, Spokane)

Rock bursts shall be defined here as: “Any sudden and violent expulsion of rock from its surroundings, the phenomenon resulting from the static stress exceeding the static strength of the rock as a result of a seismic event, or directly associated with a seismic event, and the result being of sufficient magnitude to create an engineering problem. There are no constraints on the magnitude or type of seismic event. It only needs to produce enough energy to cause damage in the tunnel.” (Obert, 1967; Ortlepp & Stacey 1994)

As pointed out in Chapter 1 rock bursts have occurred in many parts of the world. It is noteworthy that most rock burst events are concentrated in certain areas. Bennett, Marshall and Cook (1999) researched 62 areas worldwide where there have been historical reports of rock bursts. From 1995 to 1999 they recorded that 900 seismic events occurred within 50 km of 34 of these mining areas (Figure 2.5). The closed circles indicate a site with at least one event. Open circles show sites with none. One can conclude, that it is not an arbitrary phenomenon and that for tunnels too, rock bursts events should be concentrated in certain locations in the world. Unfortunately not enough data is available to confirm that, but the fact that relatively small countries like Norway and Japan stand out having repeated rock burst problems in tunnels shows that conditions in some locations favor rock bursts.

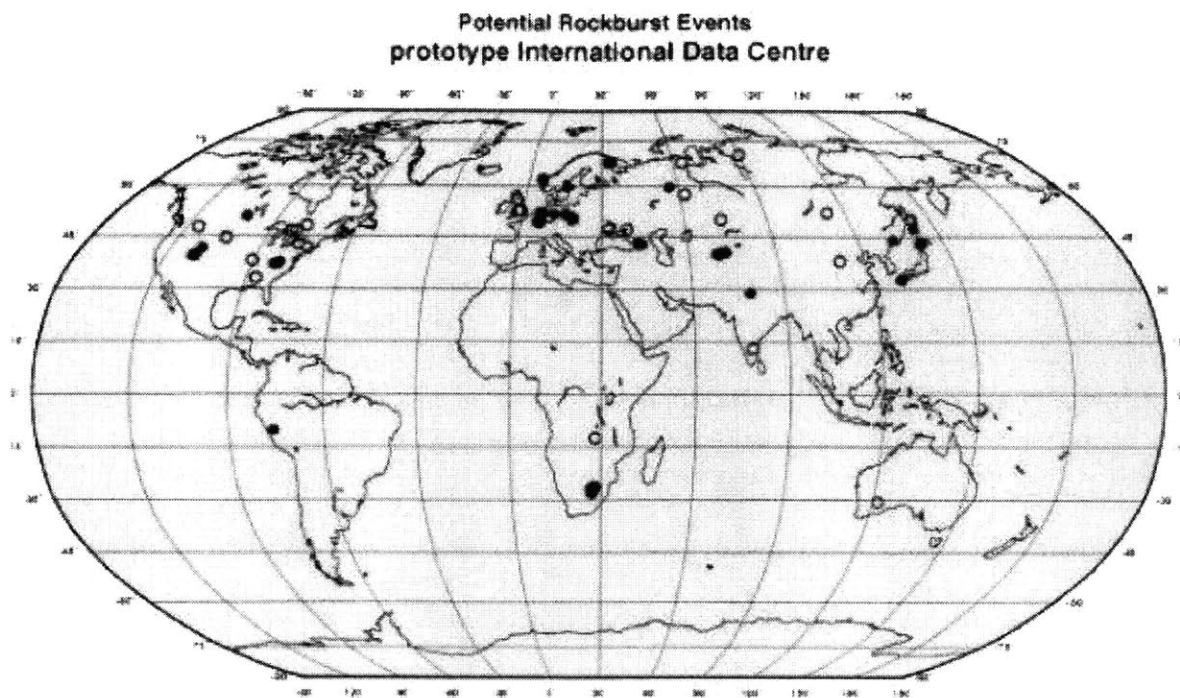


Figure 2.5: Centers of Rock Bursts Worldwide

2.1.2 Type I and Type II Rock Bursts

Two types of rock bursts can be defined and characterized. The distinction is made according to the location of the source mechanism and damage mechanism. Chapter 2.1.7 deals with mechanisms in more detail. Type I can be directly associated with the excavating process and occurs in close vicinity of the freshly excavated underground opening. It has direct effect on the tunnel and is often referred to as strain bursts, because high stress around the opening is the trigger. Type II occurs further away and involves movement on major geological discontinuities. Type II rock bursts damage the underground opening due to the rock mass displacements resulting from transmitted shock waves. The involved magnitude and damage of type II bursts can often be considerably greater than for type I bursts, if larger volumes of rock mass are involved and/or higher stresses occur away from the excavation (Vervoort & Moyson, 1997).

Table 2.1 (modified after Johnston 1988) describes both types of rock bursts for mines. It shows that type I rock bursts are triggered directly by the mining process. The existence of type II rock bursts makes the distinction between natural and induced bursts difficult. Blasting often triggers rock bursts instantaneously. Many rock bursts, particularly large damaging events in deep underground mines, occur spontaneously and unexpectedly, despite the fact that can be caused by stress changes associated with the mining process (Rohay et al, 1999).

	Type I	Type II
Rate, Trigger	Rate is a function of mining activity. Trigger is the creation of an excavation.	Not enough data to determine relationship with mining rates.
Location	Generally within 100 m of mining face or on some preexisting zone of weakness or geological discontinuity near the mine.	On some preexisting fault surface up to 3 km from the mine
Failure of intact rock or disturbed rock mass. Orientation of failure plane.	Intact rock can be broken in the rupture when mining induced stresses exceed the shear strength of the material. Orientations of rupture planes can vary.	All occur on preexisting, possibly pre stressed tectonic faults. Mining may simply trigger these events on faults of preferred orientations.
Stress change	Often high stress drops observed	Stress drops similar to natural earthquakes
Magnitude	Low to medium	Potential for high magnitudes

Table 2.1: Type I and Type II Rock Burst, modified after Johnston 1988

The fact that there is a lack of data for type II rock bursts and their association with mining activity (Johnston 1988) suggests that it can be ruled out that there will ever be enough data to link type II rock bursts directly to the excavation of normal sized tunnels, shafts or caverns. The smaller size of civil engineering underground openings is much less likely to have an impact at great distances. Moreover the seismic data acquisition for mining has been usually much more sophisticated than what is used in non-mining excavations. Rock bursts induced by excavating a civil engineering underground opening will therefore be considered to be type I rock bursts. However this does not imply that these openings cannot be affected by natural induced type II rock bursts.

2.1.3 Distinction from Other Seismic or Destructive Underground Events

To avoid confusion several other destructive mechanisms, which differ from rock bursts, are defined here. They are sometimes related to and can occur either before or after rock bursts:

- Rock falls are collapses of loosened rock to the floor of an underground excavation. In hard rock mining rock falls and other local failures caused by bumps are the reason why bumps are often mistaken as rock bursts.
- Bumps are violent failures of rock of seismic significance that do not cause damage in the underground opening. Bumps are often mentioned in the same context as rock bursts. Bumps are seismic shocks resulting from a failure or a sudden displacement at some location in the rock mass that surrounds the underground opening. The failure can be displacements along an existing fault or the shearing of an overlaying stratum. Generally the center of the disturbance is only vaguely known. The seismic shock exhibits itself acoustically or as a shock associated with a ground motion strong enough to cause partially detached rock in the roof or on the walls of underground openings to fall. In cases where the center of the bump is close enough to the underground opening the distinction between type II rock bursts and bumps becomes difficult. In addition to the bumps caused by displacements along an existing fault or the shearing of an overlaying stratum, violent pillar failures are usually referred to as either bumps or coal bursts in coal mining. The term coal burst seems to be more appropriate, since physically it is a burst due to violent failure. Even though terminology wise the term “rock burst” might not be fully appropriate for the coal environment, the physical

distinction between a bump and a burst should have preference over obvious distinctions in material. The term “burst” should therefore not be used only for bursts in non-rock environment.

- Outbursts are rapid releases of absorbed or entrapped gas as a result of rock pressure. Gas outbursts are experienced in coal-mines or salt, potash and other evaporite mineral deposits. They are usually accompanied by a violent rock failure, thus can be mistaken for bumps and rock bursts.

2.1.4 Location

For the sake of clarity principal terms will be defined. The location of the source mechanism of rock bursts, as for earthquakes, is defined by their hypocenter and epicenter. The hypocenter is the actual location of the event and is defined by the coordinates of longitude, latitude and depth. The epicenter is the surface projection of the hypocenter and does not define the depth below the surface.

2.1.5 Magnitude or Seismic Energy from Rock Bursts

The amount of energy released varies with the magnitude of the bursts. To measure the energy of local bursts (“spitting rock”) one needs micro-seismic equipment, which can measure $M_L = -0.2$ to 0 on the Richter scale. In contrast, rock bursts in mines involving large rock volume ($M_L = 2.5$ to 5), have been measured by seismological stations 1000 km away.

Theoretical solutions of how much energy is released during a rock burst are possible, if the stress and the involved rock mass volume is known. Weiss (1945) was the first to suggest that the source of the seismic energy might be the stored energy in the solid rock around the opening. Black and Starfield (1964) gave a theoretical solution for the strain energy stored in a pressed plate without a circular hole. Press and Archambeau (1962) gave the theoretical solution for the amount of seismic energy released by creation of a spherical cavity in rock subjected to a three dimensional stress field. Both Black & Starfield and

Press & Archambeau assumed as initial condition that there is no opening and determined how energy changes from this state to later condition with an opening.

The realistic condition is, however, an opening that already exists and then is suddenly enlarged. If rock bursts occurs a layer of solid rock around the opening is removed. The result is an increase in size of the opening. Duvall and Stephenson (1965) solved this for a cylindrical and spherical opening. To really understand the energy aspect of rock bursts, the derivation of Duvall's and Stephenson's solution is essential and will be shown here for the spherical cavity. This solution is for an elastic isotropic medium and cannot make provisions for structural discontinuities.

The stresses around a spherical cavity of radius a in an elastic infinite rock mass subjected to uniform radial stress $-S$ at a large distance from for spherical coordinates r , θ and ϕ from the origin is given by

$$\sigma_r = -S \left(1 - \frac{a^3}{r^3} \right); \quad \sigma_\theta = \sigma_\phi = -S \left(1 + \frac{a^3}{2r^3} \right); \quad \tau_{r,\theta} = \tau_{r,\phi} = \tau_{\theta,\phi} = 0 \quad \text{Equations 2.1}$$

where σ = normal stress and τ = shear stress in spherical coordinates
and S , a and r are defined in Figure 2.6.

Figure 2.6 shows a cross section through Duvall's and Stephenson's spherical model. The stress field illustrated is equivalent to a radial stress field with the stress S .

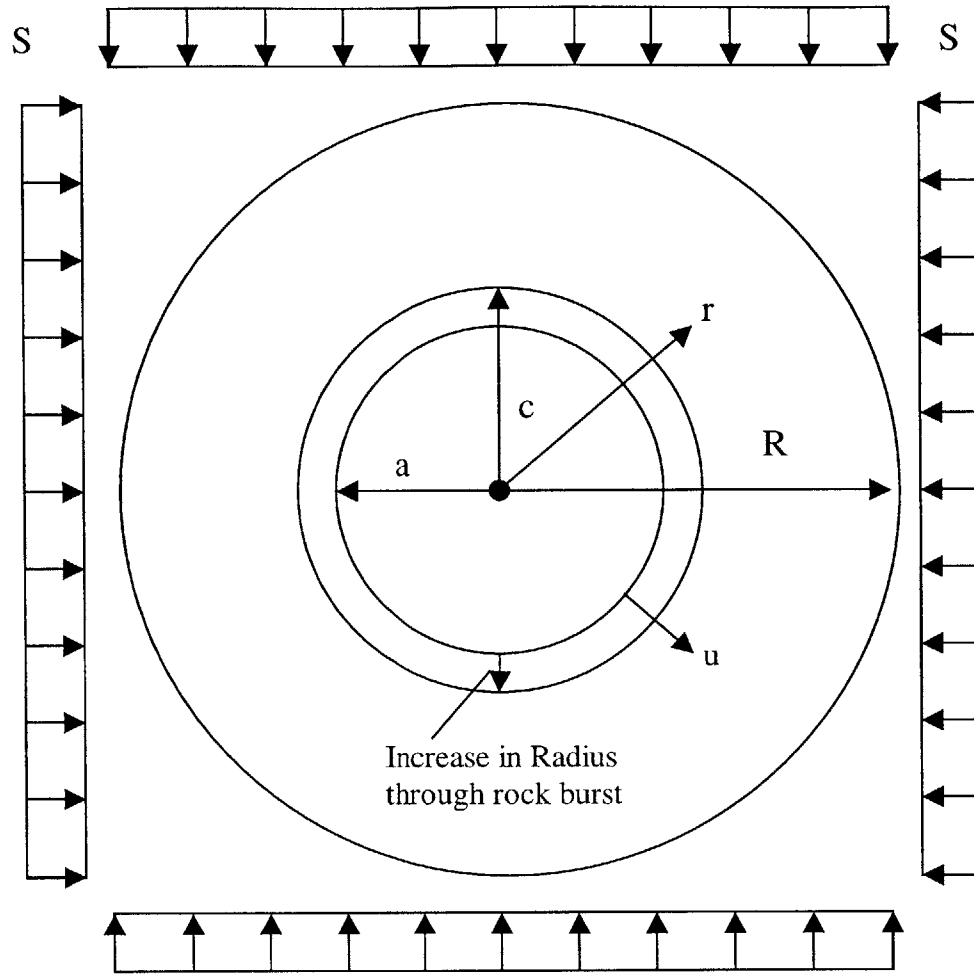


Figure 2.6: Cross Section Through Spherical Cavity Model by Duvall and Stephenson

Considering a sphere, all displacements are radial. Hooke's law and the strain displacement relationship for spherical coordinates give

$$\frac{\partial u}{\partial r} = \frac{1}{E} [\sigma_r - \nu(\sigma_\theta + \sigma_\phi)] , \text{ with } u \text{ as radial displacement.} \quad \text{Equation 2.2}$$

Substituting Equations 2.1 into Equation 2.2 gives after integration:

$$u = -\frac{S}{E} \left[(1-2\nu)r + (1+\nu)\frac{a^3}{2r^2} \right] \quad \text{Equation 2.3}$$

The strain energy per unit volume is

$$W_0 = \frac{1}{2E} (\sigma_x^2 + \sigma_y^2 + \sigma_z^2) - \frac{\nu}{E} (\sigma_x \sigma_y + \sigma_y \sigma_z + \sigma_z \sigma_x) + \frac{1}{2G} (\tau_{xy}^2 + \tau_{yz}^2 + \tau_{zx}^2) \quad \text{Equation 2.4}$$

With Equations 2.1 and Equation 2.4 one can determine the strain energy per unit volume in the rock surrounding of the cavity as

$$W_a = \frac{3S^2}{2E} \left[(1-2\nu) + (1+\nu)\frac{a^6}{2r^6} \right] \quad \text{Equation 2.5}$$

For the larger cavity with radius c, the strain energy per unit volume in the rock surrounding the cavity is

$$W_c = \frac{3S^2}{2E} \left[(1-2\nu) + (1+\nu)\frac{c^6}{2r^6} \right] \quad \text{Equation 2.6}$$

The increase in strain energy per unit volume therefore is

$$\Delta W = W_c - W_a = \frac{3S^2}{4E} (1+\nu) \left(\frac{c^6 - a^6}{r^6} \right) \quad \text{Equation 2.7}$$

A spherical shell of thickness dr has a volume of $4\pi r^2 dr$. Therefore the increase in its strain energy is $4\pi r(W_c - W_a)dr$. The total increase in strain energy ΔW_{cR} in the rock from radius c to a large radius R is:

$$\Delta W_{cR} = \int_c^R 4\pi r^2 (W_c - W_a) dr \quad \text{Equation 2.8}$$

Substituting into Equation 2.7 and integrating Equation 2.8 gives

$$\Delta W_{cR} = \frac{S^2 \pi (1 + \nu) (c^6 - a^6)}{E} \left(\frac{1}{c^3} - \frac{1}{R^3} \right) \quad \text{Equation 2.9}$$

With $R \gg c > a$ simplifies to

$$\Delta W_{cR} = \frac{3 S^2 (1 + \nu)}{4 E} \left(1 - \frac{a^3}{c^3} \right) V_{ac} \quad \text{Equation 2.10}$$

with $V_{ac} = \frac{4}{3} \pi (c^3 - a^3)$ being the volume of material removed, when the radius of the cavity is increased from a to c . As a result of removing this volume of material, the strain energy in the rock mass surrounding the cavity with the new radius c has increased by ΔW_{cR} .

The energy added to the medium surrounding the cavity by the applied stress field can be determined for any spherical surface with the radius R , the area $4\pi R^2$ and the total radial force of $4\pi R^2 \sigma_r$ acting on it. The work done by this force is

$$\Delta W_{ac} = \int 4\pi R^2 \sigma_r du \quad \text{Equation 2.11}$$

The value du is the radial displacement that takes place when the radius of the cavity is increased from a to c . It can be obtained by differentiating Equation 2.3 with respect to a .

$$du = \frac{-3S(1+\nu)a^2}{2ER^2} da \quad \text{Equation 2.12}$$

Substituting Equations 2.1 and Equation 2.12 into Equation 2.11 gives

$$\Delta W_{ac} = \frac{6S^2\pi(1+\nu)}{E} \int_a^c \left(a^2 - \frac{a^5}{R^3} \right) da \quad \text{Equation 2.13}$$

which solves to

$$\Delta W_{ac} = \frac{6S^2\pi(1+\nu)}{E} \left(\frac{c^3 - a^3}{3} - \frac{c^6 - a^6}{6R^3} \right) \quad \text{Equation 2.14}$$

which simplifies for $R \gg c$ and $R \gg a$ to

$$\Delta W_{ac} = \frac{3S^2(1+\nu)}{2E} V_{ac}, \text{ with } V_{ac} = \frac{4}{3}\pi(c^3 - a^3) \text{ being the material} \quad \text{Equation 2.15}$$

removed as the radius increases from a to c .

Equation 2.15 determines the work done by the applied stress field at a radius of R when the cavity size is increased from $r = a$ to $r = c$. Equation 2.10 determines the increase in total strain energy in the rock from $r = c$ to $r = R$. The difference between the two is the seismic energy W_s released.

$$W_s = \Delta W_{ac} - \Delta W_{cR} = \frac{3}{4} \frac{S^2(1+\nu)}{E} V_{ac} \left(1 - \frac{a^3}{c^3} \right) \quad \text{Equation 2.16}$$

simplifies for $c \gg a$ or if $a=0$ and $E = 2G(1+\nu)$ to

$$W_s = \frac{3}{4} \frac{S^2(1+\nu)}{E} V_{ac} = \frac{3}{8} \frac{S^2 V_{ac}}{G} \quad \text{Equation 2.17}$$

Duvall and Stephenson also calculated the seismic energy release for a circular tunnel. The derivation is similar to the one for the sphere shown here. The final equation for the circular tunnel is

$$W_s = \frac{1}{2} \frac{S^2}{G} V_{ac} \quad \text{Equation 2.18}$$

The small difference between the factors $\frac{1}{2}$ and $\frac{3}{8}$ suggests that the shape of the cavity does not have a significant effect on the seismic energy released during rock bursts.

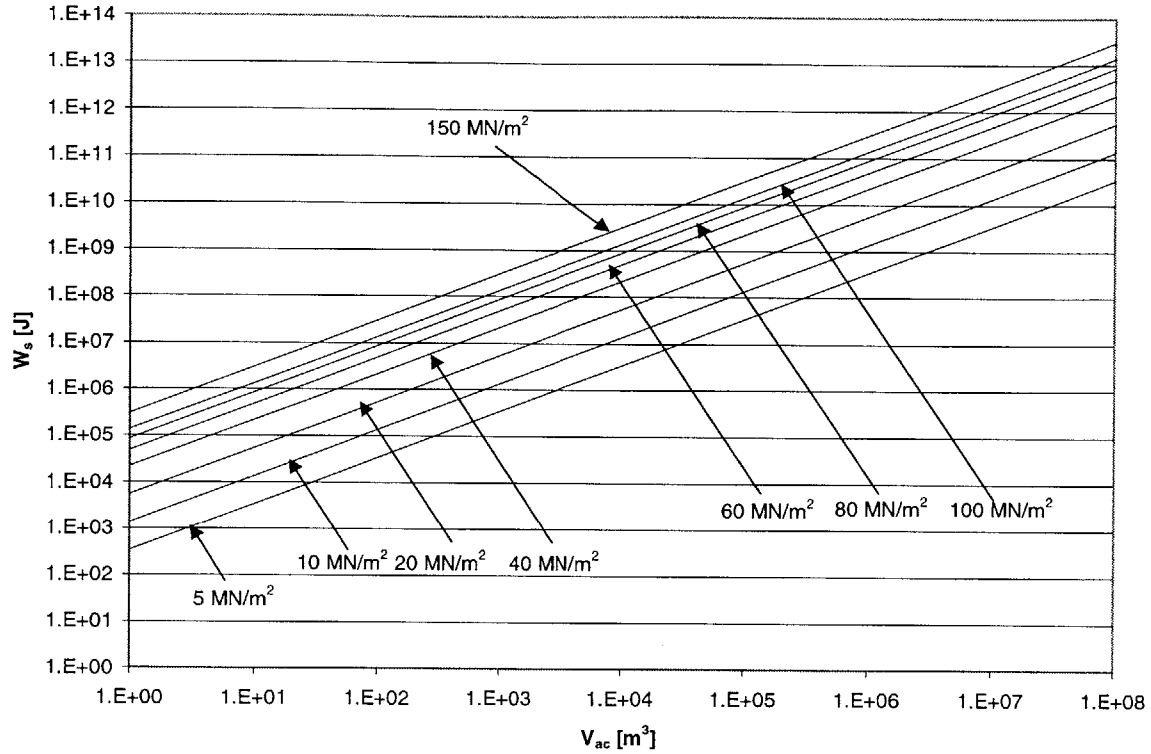


Figure 2.7: Seismic Energy W_s as a Function of Stress and Involved Rock Volume V_{ac} for Stresses from $S=5 \text{ MN/m}^2$ to $S=150 \text{ MN/m}^2$

To get an idea how much seismic energy a tunnel support has to withstand, if a rock burst occurs close to the tunnel Equation 2.18 can be normalized to the surface area of the initial tunnel. l is the tunnel length, a the radius of the initial tunnel and d the thickness of the ejected rock mass.

$$\begin{aligned}
 w_s &= \frac{1}{2} \frac{S^2}{G} \frac{V_{ac}}{A} = \frac{1}{2} \frac{S^2}{G} l \frac{(c^2 \pi - a^2 \pi)}{A} = \frac{1}{2} \frac{S^2}{G} \frac{l((a+d)^2 - a^2) \pi}{A} \\
 &= \frac{1}{2} \frac{S^2}{G} \frac{l(2ad + d^2) \pi}{A} = \frac{S^2(2ad + d^2)}{4Ga}
 \end{aligned}
 \tag{Equation 2.19}$$

2.1.6 Peak Particle Velocity and Repetitive Damage

In search for some basic parameters to characterize the damage potential of a seismic event, attention has been focused on peak particle velocity (ppv). Peak particle velocity alone is insufficient to determine the damage potential. The same ppv can cause damage of various scopes depending on the damage mechanism. Chapter 2.2 explains source and damage mechanisms in detail. It has also been found, that a single exposure to high peak particle velocities does not necessarily cause high damage. The number of impacts plays an important role as well. As can be seen from Figure 2.8 a remotely triggered rock burst is exposed to more than one shock wave. Figure 2.8 shows the back-calculated magnitude (M^*) over time of the source mechanism for a rock burst event at Creighton mine in November 1990. Particularly remarkable is the seismic activity about an hour before the main event.

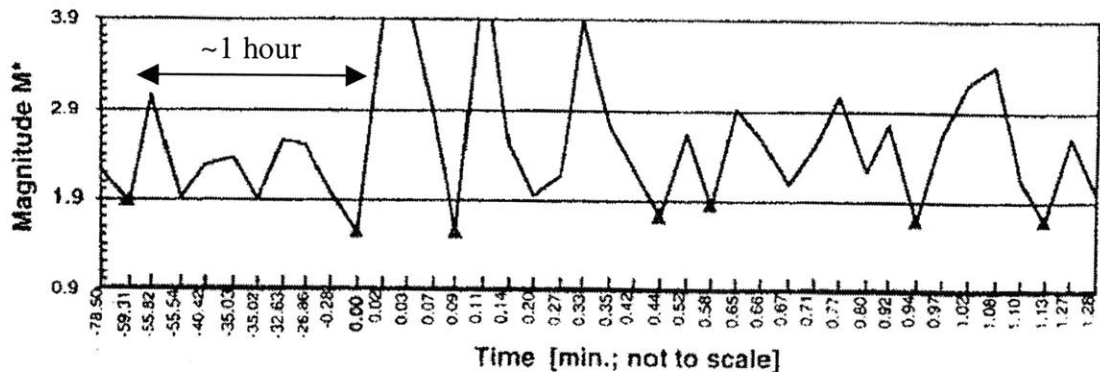


Figure 2.8: Magnitude over Time at Creighton Mine (Kaiser et al., 1992)

2.1.7 Ejection Velocity

In order to establish an energy based design approach for certain damage mechanisms in burst prone ground (Chapter 4), one needs parameters to determine the released energy. Since velocity squared is proportional to energy, the ejection velocity is an important parameter. Yi & Kaiser (1993) supply theoretical estimations of ejection velocities considering block size and peak particle velocities. Tannant, McDowell, Brummer, and Kaiser (1993) measured ejection velocities in a rock burst simulation experiment. They installed blast holes in the wall of a tunnel and triggered rock bursts with explosives. Ejection of blocks was recorded with video cameras and the ejection velocity v_e was determined from the images for single blocks and for the entire mass from ballistic trajectories (Equation 2.20).

$$v_e = D \sqrt{\frac{g}{2H \cos^2 \theta + D \sin 2\theta}} \quad \text{Equation 2.20}$$

D is the horizontal distance and H is the vertical distance the block travels and θ is the initial ejection angle measured upwards from the horizontal.

The sequence of events during the blasts was as follows: Small rock fragments were ejected from wall → Ejection of rock starts cutting through wires of velocity probes → Ejection of smaller rock blocks → Large volumes of rock start to eject → Bulk of the ejected rock in motion → Largest blocks in motion → Most ejected rock on floor → Gravity driven blocks fall to the floor.

Mass	Ve (m/s)	Number of blocks
< 5 kg	4, 4, 5, 6, 6, 6, 7, 9, 10, 22	10
5 kg to 10 kg	4, 5, 5, 6, 6, 7,	6
10 kg to 25 kg	8	1
25 kg to 50 kg	4, 6	2

Table 2.2: Summery of Velocities Estimated from video records

A summery of velocities estimated from video records is given in Table 2.2. This confirms the observation that the largest blocks, which were found closest to the tunnel wall had the lowest ejection velocities. These results confirm, what Ortlepp (1992) observed: ejection velocities of up to 10 m/s are a reasonable assumption for design but higher ejection velocities are possible.

2.2 Mechanisms

It is essential to understand that the rock burst and the seismicity is not necessarily the same event. Not only can they occur in different locations, but also at different times. A distinction between two groups of mechanisms when dealing with rock bursts is therefore necessary:

- Source mechanisms
- Damage mechanisms

The source mechanism is the mechanism that causes the seismic event, which leads to the rock burst at the location of the damage mechanism. The source mechanisms hypocenter can be far away from the location of the damage. The damage mechanism is the mechanism actually observed which causes the damage directly and its location is identical with the damage. In recent years seismological studies have provided considerable insight into the source mechanism, but studies of the mechanism of damage are less advanced (Ortlepp 1992).

2.2.1 Source Mechanisms

It has been found that the source mechanism is often controlled by the mine layout and regional structures such as faults and dykes (Durrheim et. al., 1998). It is useful to divide them into two groups. These two groups are the same as for the classification into type I and type II rock bursts mentioned earlier. To illustrate how impulsive loading on a tunnel can vary in nature and intensity, the main five source mechanisms will be discussed here briefly. Since it is the source mechanism which determines the intensity of the seismic event the listing of source mechanisms will be in ascending order of energy magnitude:

- Group A, self triggered rock bursts; source mechanism and damage mechanism probably coincide:
 - Strain bursting
 - Buckling
 - Face crushing

Strain bursting occurs at the surface and is strongly influenced by the stress concentrations of the tunnel surface. Failures due to buckling and face crushing are also by the openings in the immediate vicinity.

- Group B; remotely triggered rock bursts; source mechanism and damage mechanism most likely do not coincide:
 - Virgin shear in rock mass
 - Reactivated shear on existing faults and/or shear rupture on existing discontinuities

These mechanisms are failures on a plane and their extent could be hundreds of meters. They can be far away from the cavity and are likely to occur only in association with wide-ranging mining activities.

Ortlepp (1992) has listed all five mechanisms and lists fundamental nature and seismic magnitude (Table 2.3). They are listed in ascending order of energy magnitude and the group according to the above scheme has been added to the table. All five mechanisms have been observed to occur in mining areas. In a civil engineering environment the first mechanism is most likely to be the most common. The second and third mechanisms are less likely to occur. As mentioned earlier the fourth and fifth can be ruled out, as they only occur in association with wide-ranging mining activities.

Seismic Event	Group	Postulated Source Mechanism	First Motion from Seismic Records	Richter Magnitude M_L
Strain-bursting	A	Superficial spalling with violent ejection of fragments	Usually undetected; Could be implosive	-0.2 to 0
Buckling	A	Outwards expulsion of pre-existing larger slabs parallel to opening	Implosive	0 to 1.5
Face Crush	A	Violent expulsion of rock from tunnel face	Implosive	1.0 to 2.5
Shear rupture	B	Violent propagation of shear fracture through intact rock mass	Double-couple shear	2.0 to 3.5
Fault-slip	B	Violent renewed movement on existing fault	Double couple shear	2.5 to 5.0

Table 2.3: Classification of Seismic Event Sources (Ortlepp, 1992)

2.2.2 Damage mechanisms

Local rock conditions and support systems strongly influence the location of the damage mechanism and severity of the damage (Durrheim et. al., 1998). There is no comprehensive study published on the mechanics on rock burst damage. From the definitions available, Ortlepp's (1992) categorization into observed types of damage will be used as a guideline. According to Ortlepp (1992) damage mechanisms can be classified as:

- Strain bursting or small scale buckling (2.2.2.1)
- Buckling (2.2.2.2)
- Ejection (2.2.2.3)
- Arch collapse (2.2.2.4)
- Other: implosive type damage (2.2.2.5)

Only the strain bursting and buckling mechanism are likely to occur in a civil engineering cavity. The others are restricted to mining tunnels.

2.2.2.1 Strain Bursts

Strain bursts are the most common damage mechanism in civil engineering excavations. Fragments of rock are violently ejected locally from the rock surface. Figure 2.9 shows the typical geometric characteristics of a strain burst from a tunnel surface. Strain bursting is most likely to occur 1 to 6 tunnel radii away from the face. In some cases it was observed (Stillborg & Hamrim, 1990) to occur in the face itself. The locations in the cavity from which the fragments may be ejected vary, depending on the orientation of the in-situ stress field and the geometry of the cavity profile. In the case of the Norwegian highway tunnels it was observed (Broch & Sorheim, 1984) that the topography and its effect on the in-situ stress field determine the extent of rock bursting around the circumference of a tunnel in a single cross section (Figure 2.10).

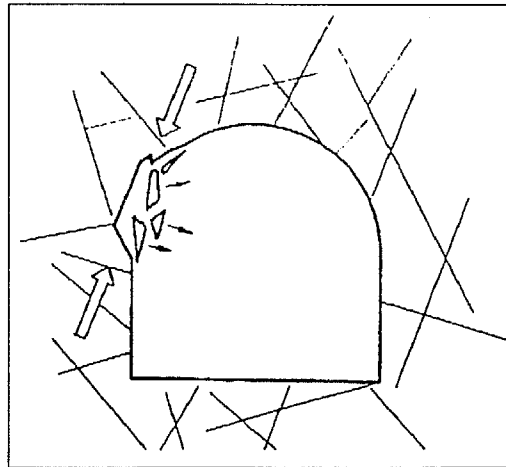


Figure 2.9: Geometry of a Strain Burst Event (Ortlepp&Stacey, 1994)

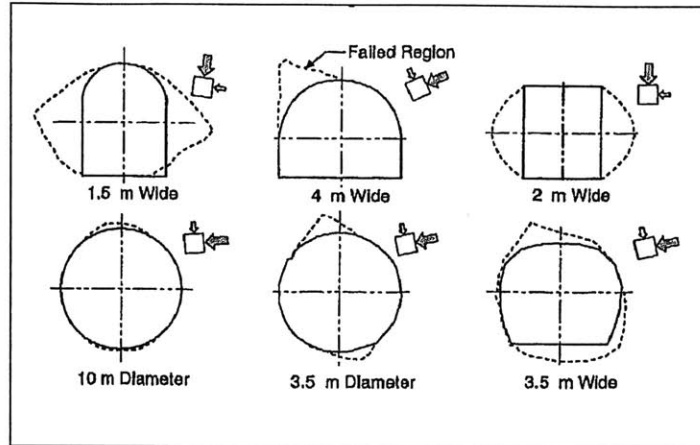


Figure 2.10: Effect of Stress Field and Cavity Profile and Damage Location (Martin et al., 1999)

The ejected fragments usually are thin plates with very sharp edges. The surfaces of the ejected fragments usually indicate extensional fracturing. It is believed that the mechanism is buckling of the thin diaphragm, slab or column rock (Ortlepp & Stacey, 1994). A typical fragment can be seen in Figure 2.11. The fragments are usually not very large, but large enough to be hazardous due to their mass. The main safety concerns arise due to their sharp edges.

All Rock burst problems described in Chapter 3, which occurred in Norway (Broch & Sorheim, 1984; Grimstad 1986; Myrvang & Grimstad, 1983), in Japan (Stillborg & Hamrim, 1990), in China (Kuitenbrouwer, L.; 1997) and in New York (Binder 1978) are classifiable as strain bursts mechanisms.

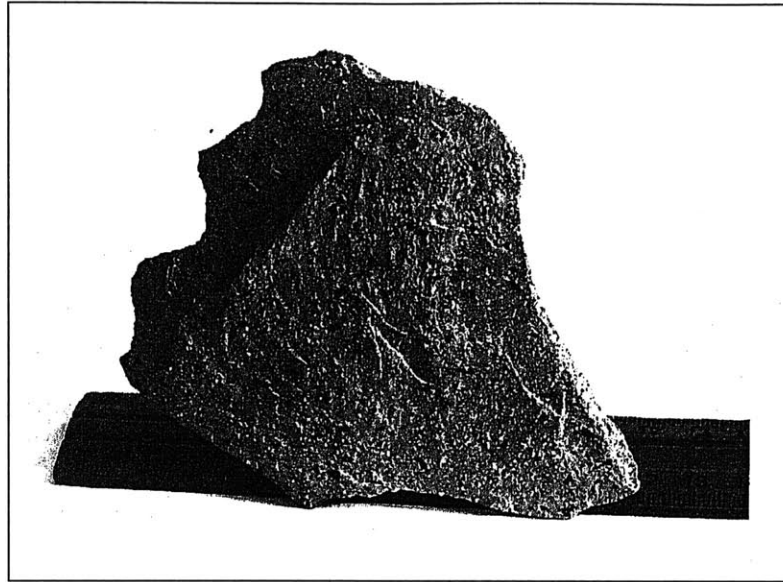


Figure 2.11: Strain Burst Fragment

The following six conclusions can be drawn from reported strain bursting events:

1. *Strain bursting is more likely to occur in more massive rock types than in heavy jointed and fractured rock.* Broch & Sorheim (1984) showed that rock bursting activity increased with increasing rock strength.
2. *Strain bursting is more likely to occur in a machine-excavated tunnel than in a tunnel where the drill and blast excavating method is used.* Stacey & Thompson (1991) reported a case of two tunnels. One was driven using a roadheader and the other one by drill and blast. In the tunnel using the roadheader violent slabbing occurred, but no slabbing was recorded for the neighboring tunnel using drill and blast. Drilling and blasting causes the rock around to fracture and consequently distresses it. The fractured distressed rock is much less burst prone.

3. *Strain bursting does not only occur in brittle rock, but is more likely to be more severe in brittle rock.* Stacey & Thompson (1991) reported strain bursting in non-brittle kimberlite. Bursts in sandstones, siltstones and shales were reported by Sperry & Heuer (1972).
4. *High stresses are not necessary for strain bursting.* Stacey (1989) estimated that fracturing in tunnels i.e. strain bursting could start at 15% of the rocks uniaxial compressive strength.
5. *Fracturing of rock in a machine-excavated tunnel can cause significant cutting problems.* High cutter wear, blocking of mucking chutes, compacting of the cutter head, high torque demand, cutter head vibration and drill string failure have been reported by Stacey (1989) and Stacey & Harte (1989) in deep-level raise boring, a boring method used in mining. Satisfactory gripper contact may be impossible to achieve in cases of severe strain bursting from the tunnel sidewalls.
6. *Encountering strain bursting conditions might significantly decrease tunneling progress rates.* Sperry & Heuer (1972) reported a decrease of up to 25%, Myrvang & Grimstad (1983) of up to 50%. Binder (1978) states that strain bursts in the face ejected about 25% of the mucking volume of a regular round.

2.2.2.2 Buckling

Figure 2.12 illustrates a rock burst buckling damage mechanism. It is most likely to occur in a laminated or transversely anisotropic rock environment. Buckling can occur anywhere around the tunnel where the orientation of the geological structure facilitates buckling instability.

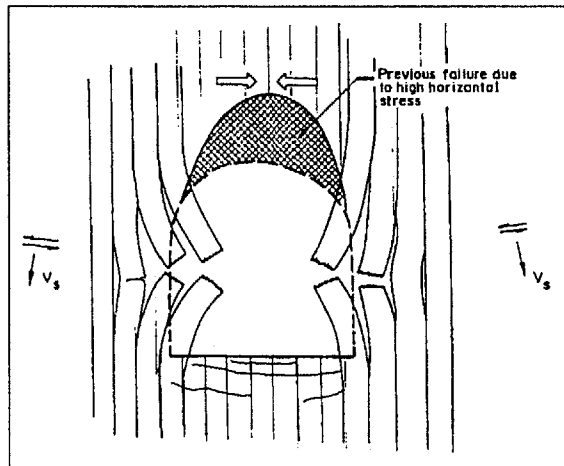


Figure 2.12: Geometry of a Buckling Event (Ortlepp&Stacey, 1994)

The slabs have buckling potential because of the strain energy, which is stored in them. A seismic shear or compression wave initiated by a distant source can trigger the failure by adding additional energy. In case of resulting instability (buckling) the locally stored strain energy will be released. Buckling has a dual nature of the energy sources causing the failure. The location of the cause of the local source mechanism (strain energy) and the damage mechanism is the same, even though the initiating source mechanism of the seismic wave can be far away.

Semadeni (1991) reports rock bursts with buckling as a damage mechanism to have taken place in the Strathcona Mine in Canada (Figure 2.13). The Richter magnitude was 1.8 to 2.3 and about 300 tons of rock were expelled from the face, roof and side walls.

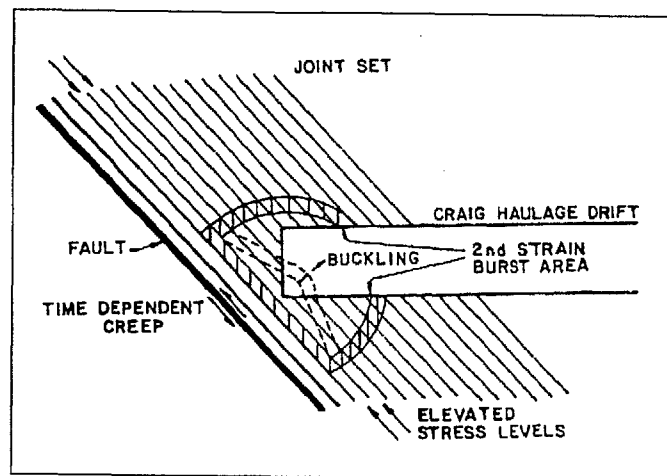


Figure 2.13: Buckling at the Strathcona Mine, Canada (Ortlepp&Stacey, 1994)

2.2.2.3 Ejection

The damage mechanism is the ejection of a part of the tunnel wall, roof or floor (Figure 2.14). It is most common in mines and manifests itself with extreme violence. Present existing joints and induced fractures determine the freedom of movement and the shape of the ejected block. The direction of ejection is closely related to the direction of the short term energy wave triggering the ejection. A seismic event is the source of energy and its magnitude and proximity to the tunnel determine the extent and violence of the damage. Its epicenter can be at some distance from the damage zone; hence the source and damage locations are not the same.

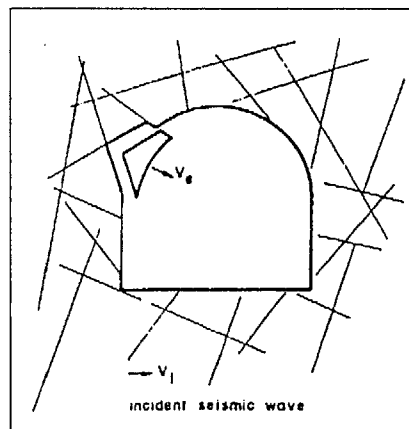


Figure 2.14: Geometry of an Ejection Event (Ortlepp&Stacey, 1994)

Damage observations in numerous mines and simulated rock burst with blasting suggest that typical ejection velocities are up to 10 m/s. Considering the substantial size of the ejected blocks they possess high energy and high damage potential. Fortunately, this type of damage mechanism is unlikely to occur in civil engineering projects, since seismicity at a distance (virgin shear in rock mass or reactivated shear on existing faults and/or shear

rupture on existing discontinuities) is the source mechanism, which is much more common in mining than civil engineering.

2.2.2.4 Arch Collapse

This type of rock burst damage mechanism is frequently observed. It can be viewed as a subcategory of the ejection damage mechanism. Figure 2.15 illustrates the geometry of an arch collapse. A well-defined geological structure or/and induced fracturing makes the kinematic movement of large blocks and wedges due to gravity easier. The trigger, again, is the seismic wave energy from a source mechanism. Its epicenter can be quite distant to the damage location. The superimposed forces from gravity and acceleration due to seismic waves lead to shearing on well-defined surfaces. It has been observed quite frequently that wedge movements shear off rock bolts, when this damage mechanism occurs.

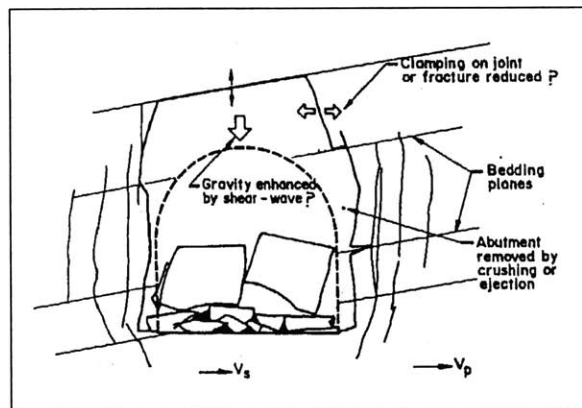


Figure 2.15: Geometry of an Arch Collapse Event (Ortlepp&Stacey, 1994)

2.2.2.5 Other Damage Mechanisms

Ortlepp (1992) describes two further damage mechanisms. They are depicted in Figure 2.16 and Figure 2.17. Implosive damage mechanisms (Figure 2.16) have been observed in mining tunnels. They result from roughly symmetrical, massive radial displacements of the fractured ring around the tunnel.

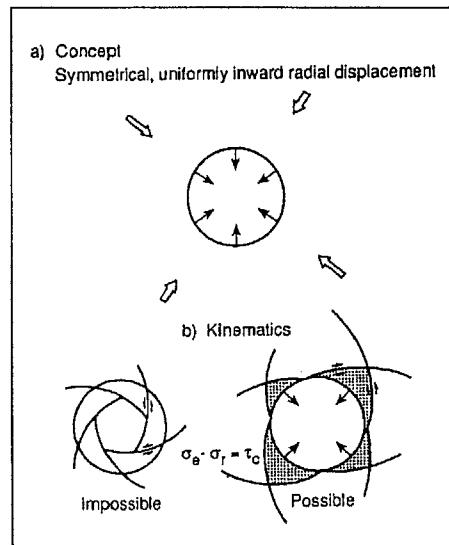


Figure 2.16: Implosive Damage Mechanism

The inertial displacement mechanism in (Figure 2.17) so far is only hypothesized. It assumes failure in the sidewall farthest away from the seismic source. The failure has low ejection velocity. One can imagine the tunnel moving from the shaking and the ejected wall remaining in place. Both, the implosive damage mechanisms and inertial displacement mechanism, are unlikely to occur in a civil engineering underground opening and are therefore not discussed further.

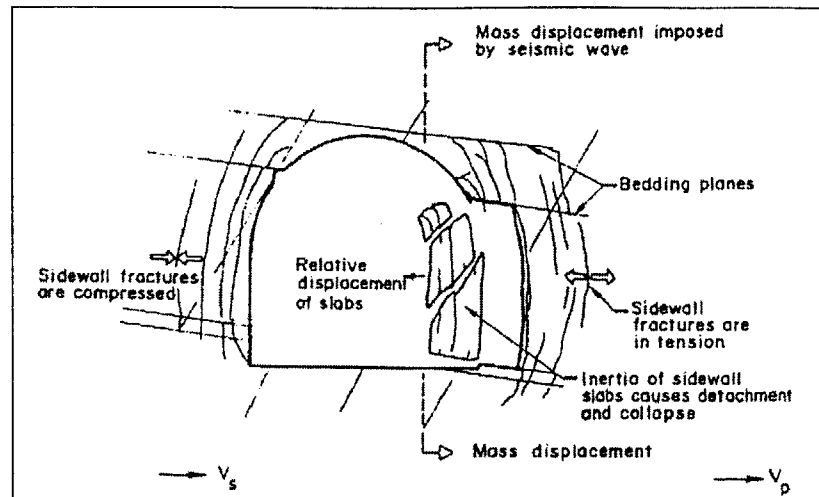


Figure 2.17: Inertial Displacement Mechanism

2.3 Depth of failure for Strain Bursts

From experience with strain bursts in tunnels it was observed that bursting will stop when a certain depth of failure is reached. Martin et al. (1998) and Kaiser et al. (1996) developed equations to determine the failure depth. The progressive failure stops when the stress drops below the rock's strength and/or a more stable excavation shape (Figure 2.18) has been created.

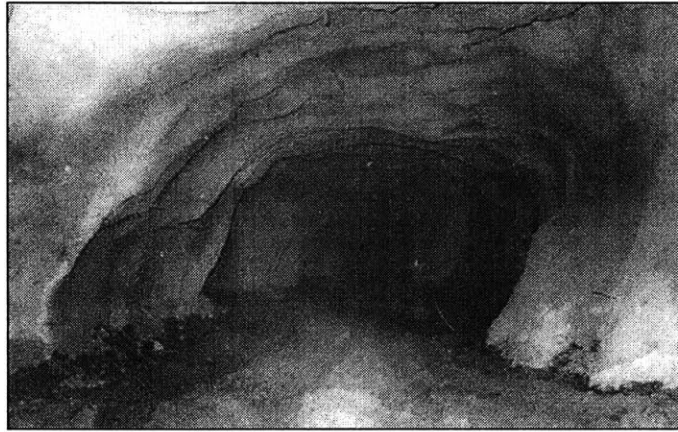


Figure 2.18: Shape after Strain Bursts Halted and Recessupport with Shotcrete (Kaiser & Tannant, 1999)

For circular tunnels, the depth of failure (d_f) is defined as the depth from the originally excavated tunnel boundary to the point where spalling stops (Figure 2.19). At this point the rock mass is self supporting under static conditions after the rock burst.

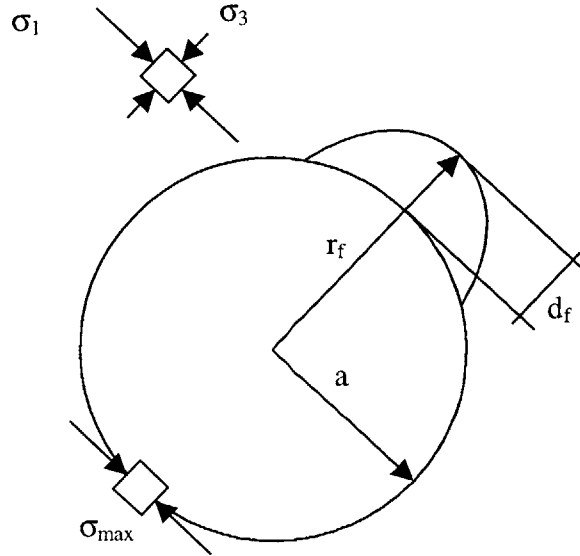


Figure 2.19: Depth of Failure

Several criteria to predict the initiation of spalling (Table 2.4) and depth of failure (Table 2.5) have been developed. The main factor is the ratio between the major principal stress in the far field (σ_1) and the unconfined compressive strength (σ_c). The higher this ratio is the more intense the spalling.

Spalling initiated when $\sigma_1/\sigma_c > 0.2$	Ortlepp et al. (1972)
$SFR = (3\sigma_1 - \sigma_3)/\sigma_c > 0.8$	Wiseman (1979)
Stable ($\sigma_1/\sigma_c < 0.1$)	Hoek & Brown (1980)
Minor Spalling ($0.3 > \sigma_1/\sigma_c > 0.2$)	
Severe Spalling ($\sigma_1/\sigma_c > 0.3$)	

Table 2.4: Criteria to Predict Spalling Initiation

Extension Strain $\epsilon > \epsilon_{cr}$	Stacey (1981)
$m = 0$ approach	Kaiser (1994)
FLAC modeling with strain-dependent cohesion and friction strength components	Vassak & Kaiser (1995)
$d_f / a = 1.34 (3\sigma_1 - \sigma_3) / \sigma_c - 0.57 (\pm 0.1)$	Kaiser et al. (1996)
$d_f / a = 1.25 (3\sigma_1 - \sigma_3) / \sigma_c - 0.51 (\pm 0.1)$	Martin et al. (1996)
$m = 0; s = 0.112$	Martin et al. (1996)

Table 2.5: Criteria to Predict Depth of Failure

Martin et al. (1996) use special Hook & Brown parameters ($m = 0$, $s = 0.112$) not as a failure criterion, but to estimate the depth of failure with linear elastic stress analysis.

There are three important aspects from the above listed studies. First that σ_1 / σ_c is the main indicator. Second that the depth of failure is finite and determinable. And third, that the static stress level largely controls the depth of failure. Therefore the impact of dynamic stresses is not great for strain bursts. The third aspect will be illustrated by an example of the research done by Kaiser et al. (1996).

Kaiser et al. (1996) plotted the ratio depth of failure to original radius (d_f / a) over the ratio maximum elastic stress at the circular opening to unconfined compressive strength ($\sigma_{max} / \sigma_c = (3\sigma_1 - \sigma_3) / \sigma_c$). The underlying equation $d_f / a = 1.34 (3\sigma_1 - \sigma_3) / \sigma_c - 0.57 (\pm 0.1)$ is included in Table 2.5. Martin et al. (1996) developed a relationship, which is also included in Table 2.5 and is almost identical: $d_f / a = 1.25 (3\sigma_1 - \sigma_3) / \sigma_c - 0.51 (\pm 0.1)$.

In addition to the static case (Kaiser et al.) added dynamic loading conditions for different peak particle velocities (ppv) to the graph. Figure 2.20 shows the case for $\sigma_1 = 30$ MPa and $\sigma_3 = 15$ MPa ($\sigma_{\max} = 75$ MPa) and peak particle velocities (ppv) from 0 to 3 m/s.

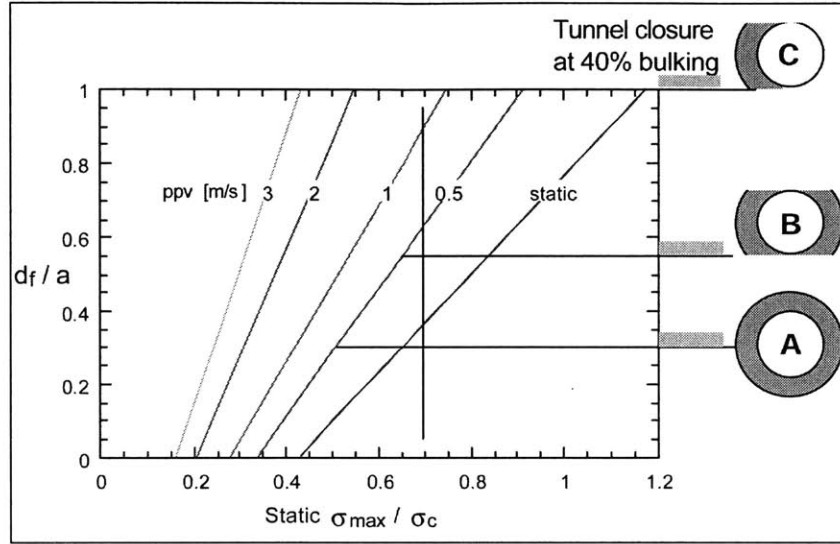


Figure 2.20: Depth of Failure under Static and Dynamic Stress (Martin et al. 1996)

The horizontal lines represent a boundary for the maximum depth of failure for all around failure (A), two-sided failure (B) and one-sided failure (C). Figure 2.20 can be used to demonstrate that for strain bursts in tunnels it is mostly the static stress level that determines the depth of failure. In our example we will use a static stress level $\sigma_{1\max}$ of 0.7, which is represented by the vertical line. Hence depth of failure for the static case would be $d_f = 0.35a$. For $\text{ppv} = 0.5$ m/s depth of failure is $d_f = 0.65a$ and for $\text{ppv} = 1$ m/s depth of failure is $d_f = 0.9a$. However, peak particle velocities in this order are encountered near strong earthquakes or fault slip bursts. Seismic waves from strain bursts are usually in the order of 0.01 m/s (Kaiser et al., 1996).

Thus Figure 2.20 demonstrates that the static stress level is the most dominant factor in determination of the depth of failure for tunnels prone to strain bursting. Dynamic stresses can have some effect in deepening the failure zone, but it is rather small unless unusual high ground motion is present. This is unlikely for most tunnels in non-earthquake areas. Therefore it can be assumed that d_f is determined by a linear function controlled by (σ_{\max}/σ_c) .

2.4 Causes of Rock Bursts

Causes of rock bursts should not be confused with source mechanisms. Causes, as the name suggests, are the cause of the source mechanism. One can distinguish between two groups of direct causes of rock bursts. Naturally induced and artificially induced. Among the natural causes are:

- Earthquakes
- Volcanic activity and movement of magma
- Tidal or flood loading or unloading
- Stress redistributions along faults
- Glacial loading or unloading

Artificial causes can be:

- Excavation or mining
- Reservoir loading

Naturally induced rock bursts can occur in both, an existing or an underground opening being constructed. Most artificially induced (excavating or mining) rock bursts occur while an opening is created or around fairly new underground openings.

2.5 Factors Influencing the Severity of Rock Bursts

Once failure is triggered, the severity of the damage depends on two main aspects on the active side: The volume of rock involved in the failure process and the energy that is released during the failure process. The latter provides a measure of the violence of failure. The volume of failing rock depends on the extent of the excessively stressed zone of rock around an excavation or the depth of failure. Factors influencing these aspects are discussed in more detail below. On the passive side probably the most important factors are the conditions of the rock mass and inadequate support systems. This is discussed in Chapter 4.

For civil engineering structures, ignoring type II rock bursts, one can generally state that an unconfined compression test serves as a good analogy: Stresses in the rock mass are close to the rocks strength and failure becomes fiercer with increasingly brittle rock.

2.5.1 In Situ Stress Condition

In situ stress is an important factor in designing underground excavations. It has been observed for a long time and is a well-known fact that both the severity of rock bursts and their frequency increase with depth. Increasing weight of the overburden and therefore increasing stresses in the rock with depth are the cause. In addition rock is more competent with depth and its ability to store significant amounts of energy increases. However, some cases have shown that this cannot be the only contributing factor. Small bursts have been reported in shallow tunnels (Norway) and larger bursts at depths of less than 300m were

reported in shallow mining. For deep tunnels or mines $k = \sigma_h/\sigma_v = 0.5$ cannot be simply assumed. In some deep mines significant residual stresses have been found to exist producing k ratios up to 1.8 (Durrheim). In any case measurements should be made to determine if any anomalous state of stress exists.

Near surface bursts are certainly caused by high horizontal stresses. These in turn can be caused by tectonic forces acting approximately parallel to the surface. In one extreme case horizontal stresses of several MN/m^2 were observed within 20 m depth in a granite gneiss quarry in Lithonia, Georgia. The Lithonia Gneiss in the area southeast of Atlanta is also known as Migmatite - a very high grade metamorphic rock that has been subjected to such high temperatures that it has partially melted. It is intermediate between metamorphic and igneous rocks.

In deep mines bursts are first experienced at about 700 m depth, but do not become a problem until depths of 1000 m are exceeded. However many mines have operated without any bursts at depths of more than 2000 m. This strongly indicates that other factors than depth must influence rock burst activity.

2.5.1.1 Interpretation of Results from Numerical Modeling

Numerical modeling is more and more frequently used to determine the stress conditions. During rock burst investigations it is frequently found, that the fundamental assumptions and results of standard elastic modeling need to be applied with a good amount of judgement.

A few lessons learned from the application of numerical modeling in mining should also be considered in civil engineering (Durrheim1998):

- Elastic modeling programs do not take the fracturing of the face into account. They produce unrealistically high values for of stress at the edges. In practice these areas fracture and crush. As a result the load gets shifted away from the surface to stiffer regions further away from the opening. However, these regions are associated with low values of stress in the numerical analysis.
- In seismic active areas, the seismic history should be considered when assigning strengths to blocks of rock. Blocks, which have been exposed to significant seismicity should not be modeled as solid blocks.
- It should be checked if the mesh size is small enough to take into account local tunnel geometry in adequate detail. The window size has to be large enough to take in account all significant stress contributions in the area of interest.

2.5.2 Mechanical Characteristics

Other than stress the second most important factor is most likely mechanical characteristics of the rock. Hard, strong, and brittle rock is most likely to be burst prone. Unconfined compressive strength of burst prone rock is in the range of 36 MN/m² to 350 MN/m² (Table 2.6). The modulus of elasticity ranges from 40,000 MN/m² to 90,000 MN/m² (Obert & Duvall, 1967). Data on how other properties like hardness or shear strength relate to rock bursts is rare.

Rock mass	R_f/a	σ_1/σ_3	σ_3 (MPa)	σ_c (MPa)	Reference
Blocky andesite ^a	1.3	1.92	15.3	100	GRC field notes (El Teniente Mine)
	1.5	2.07	14.8	100	
	1.4	2.03	14.7	100	
	1.5	2.10	16.3	100	
	1.5	2.03	15.4	100	
	1.6	2.09	15.8	100	
Massive quartzites ^a	1.8	2.15	65	350	Ortlepp and Gay (1984)
	1.7	2.15	65	350	
	1.4	1.86	60	350	
	1.5	1.86	60	350	
Bedded quartzites	1.4	3.39	15.5	250	Stacey and de Jongh (1977)
	1.3	3.39	15.5	250	
Massive granite	1.5	5.36	11	220	Martin et al. (1994)
	1.4	5.36	11	220	
	1.4	5.36	11	220	
	1.3	5.36	11	220	
	1.3	5.36	11	220	
	1.0	3.7	11	220	
Massive granite	1.1	1.31	40	220	Martin (1989)
Interbedded siltstone–mudstone	1.4	2.0	5	36	Pelli et al. (1991)
Bedded limestone	1.1	1.3	12.1	80	Jiayou et al. (1989)
Bedded quartzites	1.0	1.69	21	217	Kirsten and Klokow (1979)
	1.08	1.69	20	151	

^a D-shaped tunnel.

Table 2.6: Summary of Case histories by Martin et al., 1999

It is noteworthy, that rocks in the same stress environment and with the same unconfined compressive strength were found to be burst prone in some cases and not at all in other cases. The same was experienced for rocks with the same modules of elasticity. This implied, that there must be another property that has an effect on the likeliness of rock bursts. It was found that the higher the strain energy that can be stored in a rock type the higher its tendency to burst. Strain energy per unit volume of rock can be described as: $\sigma_c^2/(2E)$. The maximum strain energy storage capacity therefore is $\sigma_c^2/(2E)$ with σ_c being the uniaxial compressive strength. Thus all other properties being equal, this relationship again suggests, that the rock with the highest compressive strength is most likely to burst. In the deep copper mines of Michigan this relationship was confirmed: High strength glassy basalts with a compressive strength from $\sigma_c = 100 \text{ MN/m}^2$ to $\sigma_c = 200 \text{ MN/m}^2$ were much more likely to burst than low strength amygdaloidal basalts with a compressive strength from $\sigma_c = 35 \text{ MN/m}^2$ to $\sigma_c = 100 \text{ MN/m}^2$. Since the modulus of elasticity usually increases with increasing σ_c and only enter the equation linearly it does not have as much impact.

Non-bursting rock more than bursting rock seems to plastically and/or visco-elastically deform under stress so that failure takes place slowly. Unfortunately this cannot be described by a single mechanical property. A testing device and procedure was developed by Denkhaus and Grobbelaar (1955). It measures what they describe as "relative violence of rupture". The testing device measures the impulsive rebound of the bed of the testing machine, produced when a compressive strength test specimen fails. For specimens that fail more violently, it is assumed that the rebound will be greater. This test is dependent on

the mechanical characteristics of the testing machine as well as of the specimen and therefore has arbitrary units.

Table 2.7 shows the mechanical properties for 16 different rocks from research in South Africa by Hill and Denkhaus (1961) and the values of obtained in for the "relative violence of rupture" test. From the data in Table 2.7 by Hill and Denkhaus (1961) I produced Figure 2.21 to Figure 2.25 to make a comparison and the finding of a relationship easier.

Properties of Some Witwatersrand Rock (Hill and Denkhaus, 1961)						
	Unit Weight	Uniaxial Compr. Strength	Modulus of Elasticity	Poisson's Ratio	Relative Violence of Fracture	Max. Strain Energy p. Unit Volume
Type of Rock	[kN/m ³]	[MN/m ²]	[MN/m ²]	[-]	[-]	[kJ/m ³]
Shale	26.87	227	70,300	0.26	1.9	366
Shale (normal to bedding plane)	27.06	170	75,800	0.25	0.8	191
Shale (parallel to bedding plane)	27.23	205	103,400	0.20	1.4	203
Kimberley Shale	27.99	168	77,900	0.47	0.7	180
Jeppeshtown Shale	27.85	123	83,400	0.38		90
Hanging Wall Quartzite	26.42	283	82,000	0.15	4.1	487
Reef Bands	27.15	296	88,900	0.17	5.6	494
Footwall Quartzite	26.70	232	83,400	0.20	3.0	324
Main Bird Quartzite	26.62	244	81,400	0.26	3.4	366
Kimberley Quartzite	25.90	396	80,000	0.18	8.1	982
Dolomite	28.08	403	97,200	0.36	8.4	837
Chert	25.80	476	84,100	0.26	18.5	1346
Diabase (fresh)	28.74	445	105,500	0.27	15.3	940
Diabase (slightly decompo:	27.20	244	68,900	0.23	2.7	432
Porphyrite Lava	27.75	419	91,000	0.29	9.7	962
Amygdaloidal Lava	27.12	258	84,100	0.38	3.1	395

Table 2.7: Properties of Some Witwatersrand Rock (Hill and Denkhaus, 1961)

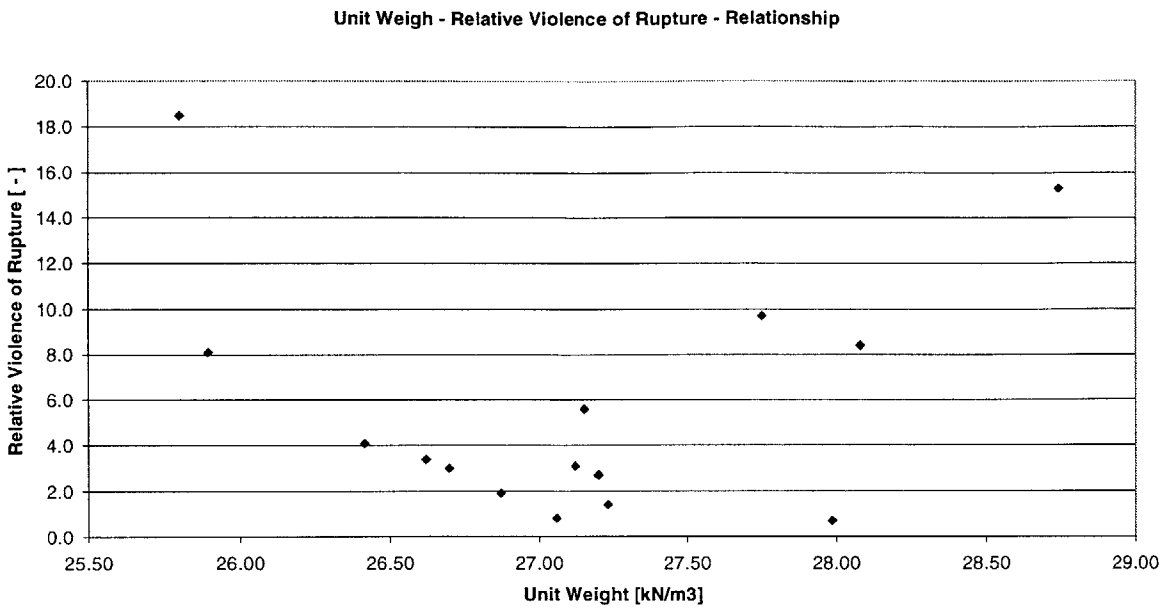


Figure 2.21: Unit Weigh - Relative Violence of Rupture - Relationship

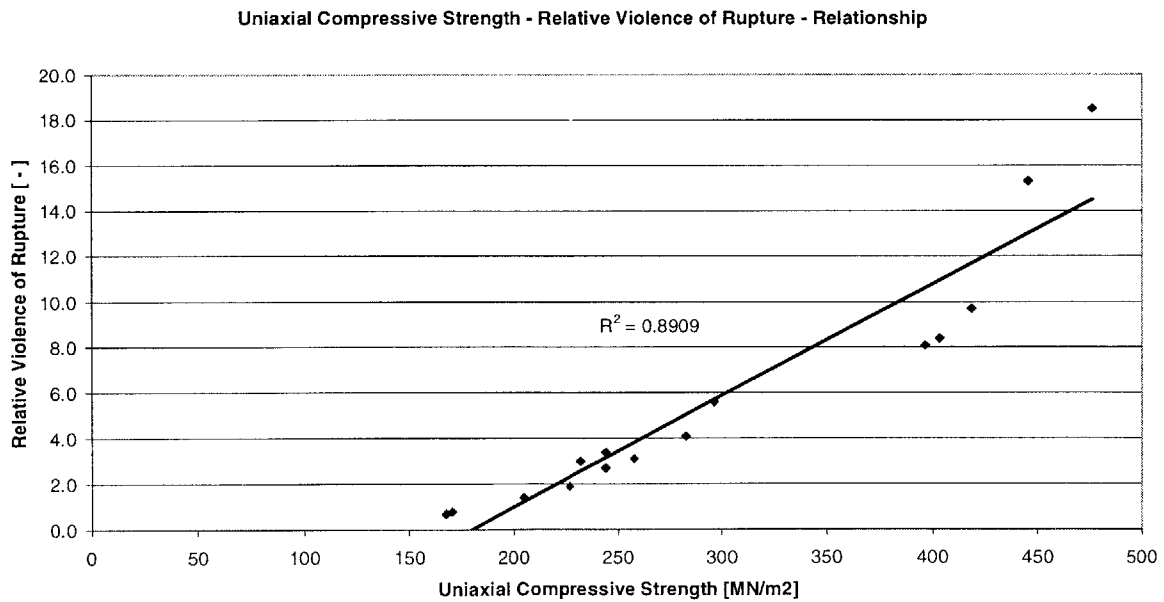


Figure 2.22: Uniaxial Compressive Strength - Relative Violence of Rupture - Relationship

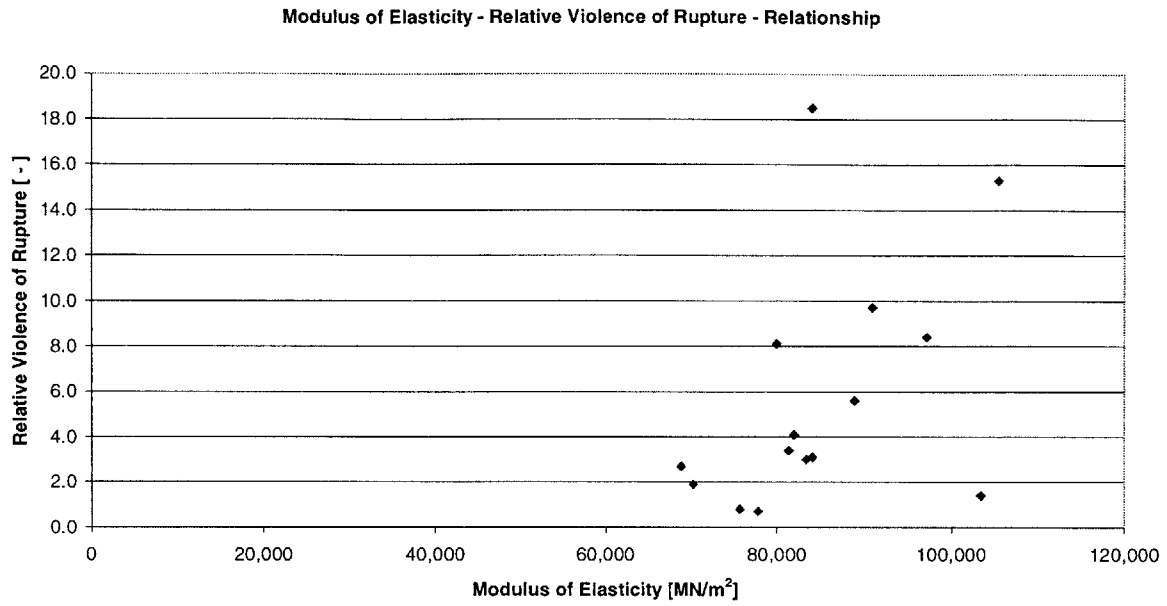


Figure 2.23: Modulus of Elasticity - Relative Violence of Rupture - Relationship

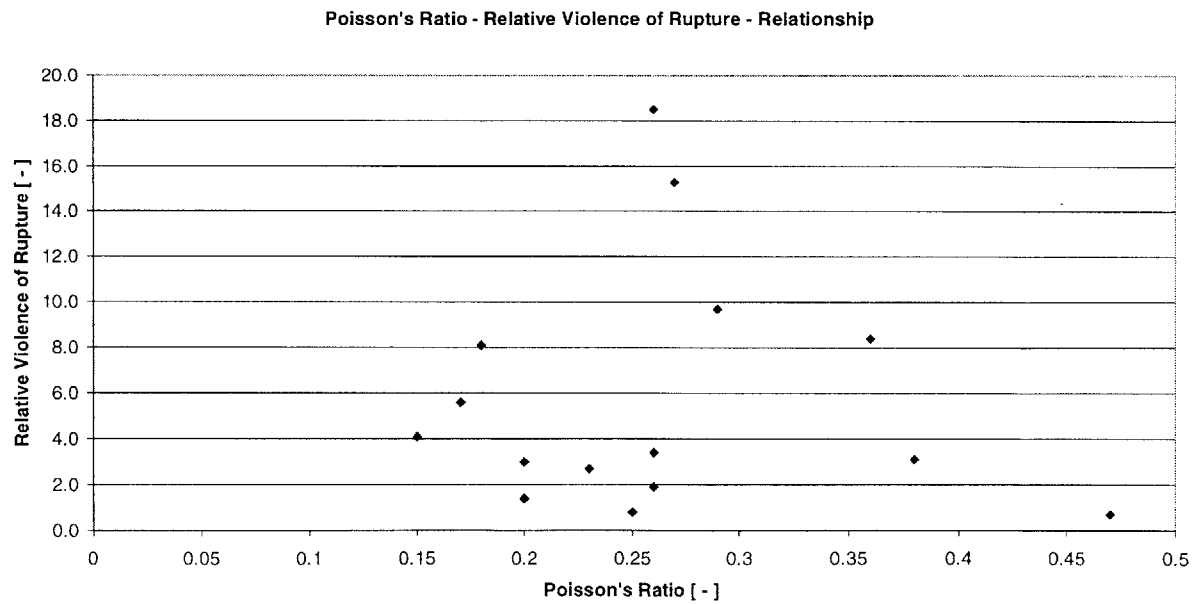


Figure 2.24: Poisson's Ratio - Relative Violence of Rupture - Relationship

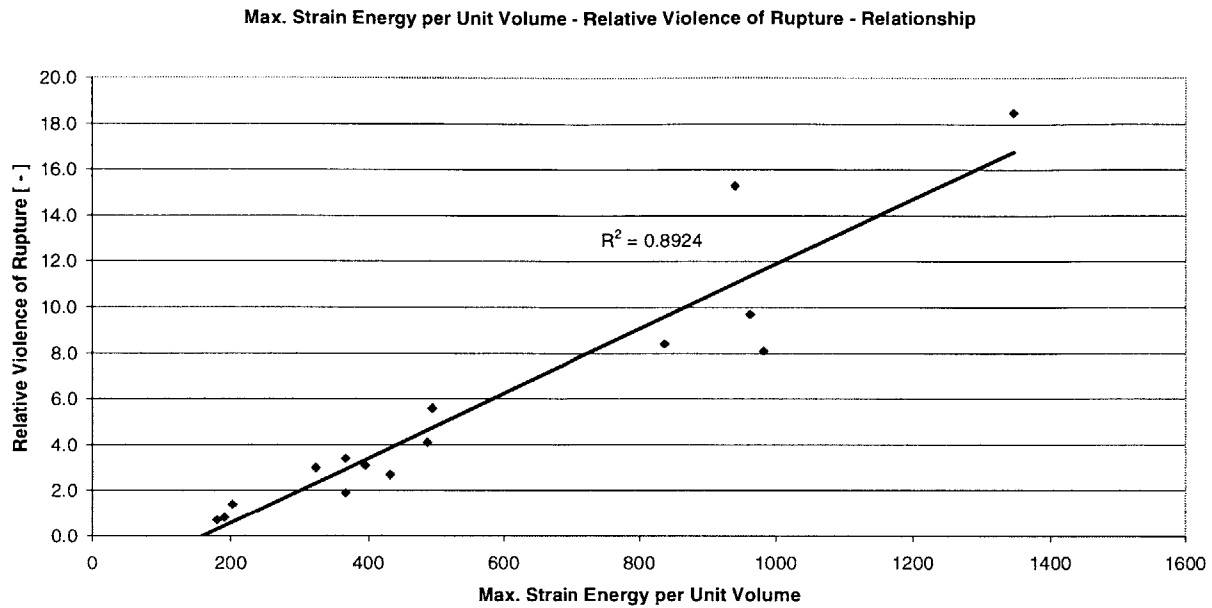


Figure 2.25: Max. Strain Energy per Unit Volume - Relative Violence of Rupture - Relationship

If one defines the "relative violence of rupture" value as the attribute of rock that defines the likeliness and severity of rock bursts in situ, the following conclusions can be drawn:

- Unit Weight (Figure 2.21): There is no clear relationship. Burst prone rock can be found at both the high and the low end of the unit weight scale. Rocks with the same unit weight can differ tremendously in their "relative violence of rupture".
- Uniaxial Compressive Strength (Figure 2.22): An almost perfect linear relationship between the uniaxial compressive strength and the likeliness to be burst prone exists ($r^2 = 0.89$). Rock with high uniaxial compressive strength is extremely burst prone and will burst violently.

- Modulus of Elasticity (Figure 2.23): No clear relationship exists, but rock with a low (below 80,000 MN/m²) modulus of elasticity seems unlikely to burst.
- Poisson's Ratio (Figure 2.24): The data do not indicate any relationship between the Poisson's ratio and the “relative violence of rupture”.
- Max. Strain Energy per Unit Volume (Figure 2.25): The maximum strain energy that can be stored per unit volume of rock was expected to have the most direct relation to the burst severity. The data show a clear linear relationship ($r^2 = 0.89$). Rock, that can store significant amounts of energy are most likely to burst with great severity.

It is worthwhile noting that the three rocks that seem to be most burst prone (Chert, Diabase (fresh) and Porphyrite Lava) are also two of the three rocks, that can store the most strain energy and have the three highest uniaxial compressive strengths, whereas only the Diabase (fresh) is among the three rocks with the three highest moduli of elasticity.

Three rocks with highest (descending order)	
Relative Violence of Fracture:	Chert, Diabase (fresh), Porphyrite Lava
Max. Strain Energy per unit Vol.:	Chert, Kimberley Quartzite, Porphyrite Lava
Uniaxial Compressive strength:	Chert, Diabase (fresh), Porphyrite Lava
Modulus of Elasticity:	Diabase (fresh), Shale, Dolomite

Table 2.8: Relation of High Rock Properties

The three rocks that seem to be most unlikely to be burst prone (Jeppestown Shale, Kimberly Shale, Shale (normal to bedding plane)) are also the three rocks, which can store the least strain energy and have the three lowest uniaxial compressive strengths.

Three rocks with lowest (ascending order)	
Relative Violence of Fracture:	Jeppetown Shale, Kimberly Shale, Shale (normal to bedding plane)
Max. Strain Energy per unit Vol.:	Jeppetown Shale, Kimberly Shale, Shale (normal to bedding plane)
Uniaxial Compressive strength:	Jeppetown Shale, Kimberly Shale, Shale (normal to bedding plane)
Modulus of Elasticity:	Shale, Shale (normal to bedding plane), Kimberly Shale

Table 2.9: Relation of Low Rock Properties

From the relationships in Figure 2.22 and Figure 2.25 and the good indications related to the three lowest and highest values, one can conclude that both, uniaxial compressive strength and the maximum strain energy storage capacity per unit volume, are good indicators of rock bursting behavior. As far as one can conclude from these data the uniaxial compressive strength can be preferred over the maximum strain energy storage capacity per unit volume, since it only requires one value to be obtained in a test and no further calculations.

2.5.3 Impact of Petrology

Brittleness is a characteristic very closely related to bursting. It can be more closely related to petrology than other mechanical properties of the rock. Some general guidelines can be found:

- Igneous and metamorphic rocks are generally more burst prone than the sedimentary rocks.
- Mineral composition: Silicious rocks and rocks containing other hard minerals are more burst prone than rocks containing carbonates and other soft minerals.
- Grain size: The smaller the more likely the burst. Bursting increases from coarse to microcrystalline to glassy or amorphous.
- Rock composition: Local stress concentrations due to frequent changes between intact rock and schistose rock intensify burst proneness.

2.5.4 Impact of Geologic Features

Hill and Denkhaus (1961) investigated the occurrence of rock bursts near faults and dykes. Dykes are “wall-like intrusive igneous rocks filling fissures”. Figure 2.26 shows how rock bursts increase as the face comes closer to a dyke and decrease after the face has passed the dyke. Notable is the almost symmetrical graph, which indicates that the passing face does not significantly distress the dyke. The data was obtained for all dykes wider than 8 m from the mining district Witwatersrand, South Africa.

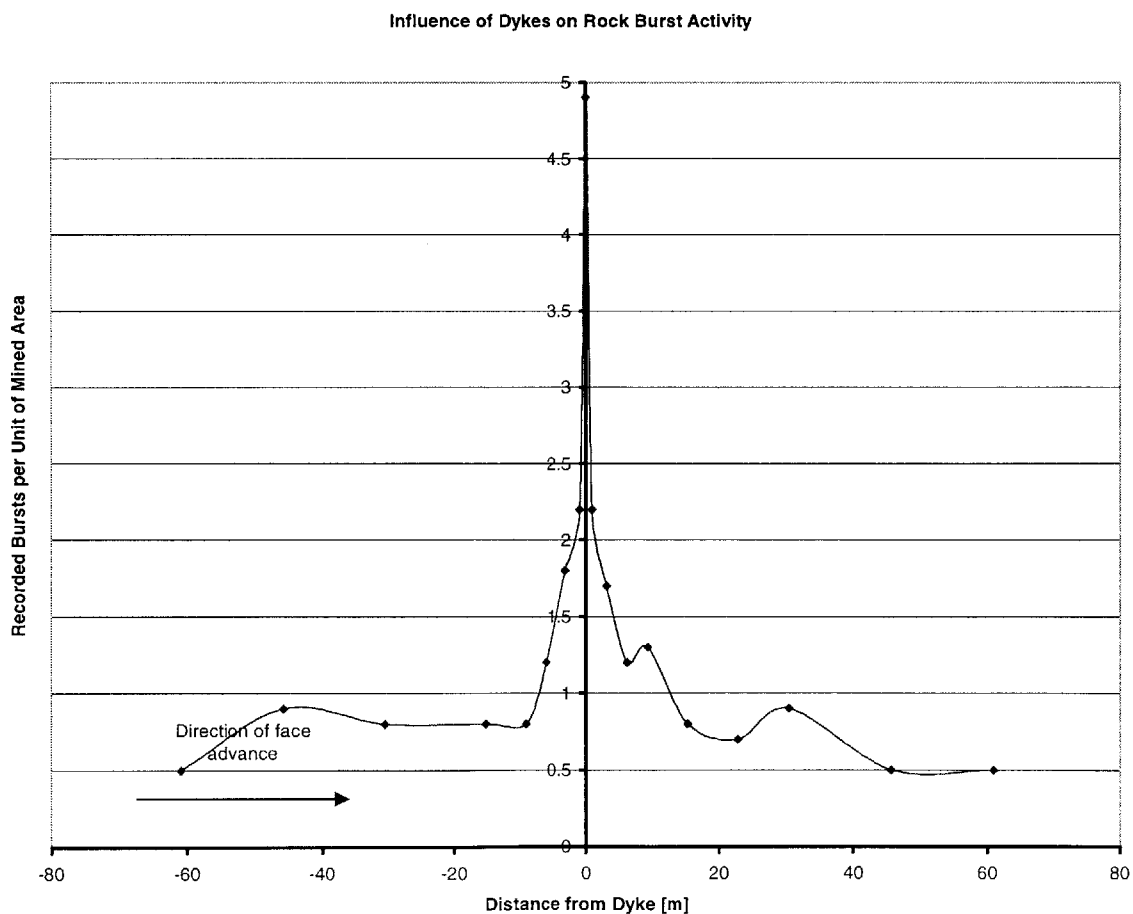


Figure 2.26: Influence of Dykes on Rock Burst Activity

The same graph can be produced for the face passing a fault (Figure 2.27). Again the influence of the fault significantly increases the number of rock bursts, but after the face has passed the fault rock bursts cease to a normal level.

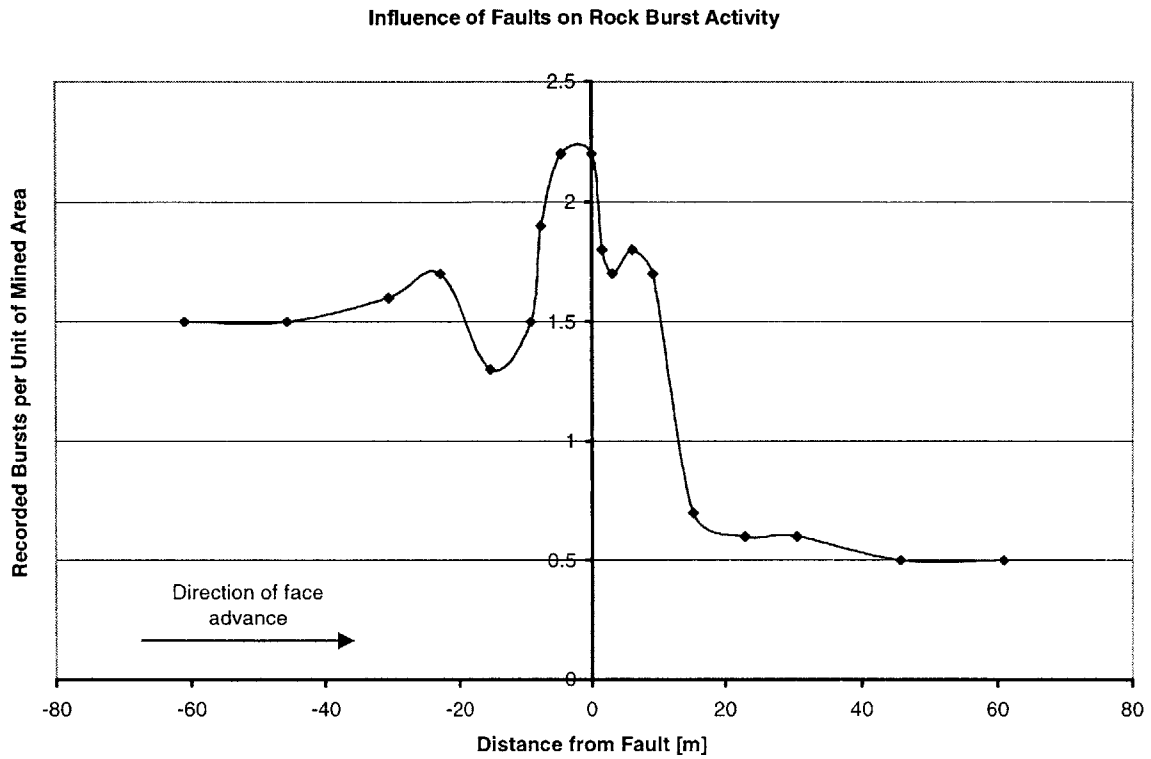


Figure 2.27: Influence of Faults on Rock Activity

2.5.5 Preconditioning

Durrheim et. al. (1998) investigated several mines where preconditioning or destress blasting was implemented. These investigations supported the view that preconditioning reduces the hazard of rock bursts. However the effectiveness of preconditioning was found to diminish with time. Therefore it is important that preconditioning blasts are based on elapsed time and not merely on face advance. Production personnel must adhere to the preconditioning schedule. Destress blasting is discussed further in Chapter 5.

2.5.6 Extraordinary Factors Influencing Rock Bursts

Earthquakes: During the construction of the underground excavations of the Ertan Dam in China several earthquakes were recorded. The most severe was recorded at 4.5 on the Richter scale and occurred in September 1995. About 12 hours after the quake, rock bursts occurred in several excavated areas.

2.6 Impact of Rock Bursts

The type of rock burst that creates major engineering problems occurs in or near working areas. Unfortunately these bursts near the working areas are the most common. They involve rock masses of up to thousands of tons. The smaller bursts usually occur in smaller underground openings such as tunnels and shafts. The larger ones were observed in extensively mined areas. Infrequently, rock burst in abandoned parts of mines and at distances far from the working areas in tunnels and shafts are observed. Although they do not directly cause any danger to personal or equipment, they can affect the overall stability of an underground opening, especially a mine.

2.6.1 Workmen Safety and Machinery

More safety related research on rock bursts is available for mines than tunnels. One Canadian report prepared by the Ontario Rockburst Committee Mining Association illustrates the magnitude of safety issues. Over a seven year period a survey in four districts (Sudbury, Lakeshore, Timmins and Little Long Lac) 1167 bursts were recorded. 115, about 10%, of these were rated as heavy or strong. The bursts caused 21 fatalities, equal to 7.5% of the total fatalities and 64 nonfatal compensation cases. Similar numbers were reported for the gold mines in Witwatersrand, Africa. Over a seven year period 9% of the total fatalities were due to rock bursts. An extreme case is the Kolar gold field in India, where over a three-year period 50% of the total fatalities were caused by rock bursts.

2.6.2 Tunnel Structure and Support

Damage on support is discussed and illustrated in Chapter 4.

2.6.3 Effects on Project Schedule and Cost

In deep mining, the loss of production during the clean up period and the time needed for building the new support together with the cost of building the extra ground support makes rock bursts a key issue in the operations of deep mines. Bursts in shallow mines occur infrequently and are not of sufficient intensity to create a major operational problem. For tunnels and shafts, where the objective is to build an opening rather than to extract material things change. Generally rock bursts are less frequent than in mines. The main cost factor is due unexpected delays in construction. Nevertheless the need for a change in procedure and equipment also affects schedule and cost negatively. Additional expenses for support might be incurred.

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Chapter 3 Case Studies

3.1 Rock Bursts in Civil Engineering Structures

3.1.1 Norwegian Highway Tunnels

Many tunnels are needed in the Norwegian Fjord areas. The rock encountered at the Norwegian fjords is mostly hard, intact and high strength precambrian gneissic rock. Lining is omitted in most cases. One drawback of the encountered brittle rocks with high compressive strengths is their susceptibility to rock bursts. Rock bursts occurred during the construction of “recent” as well as “traditional” routing. The distinction between these two cases is valuable to demonstrate how the virgin principal stress pattern influences rock burst. The difference in routing is explained below and shown in Figure 3.1.

- Traditionally, roads and tunnels (Heggura tunnel in Tafjord) were located parallel and around the fjords through the steep mountainside. In many cases avalanche galleries had to be built in addition to the tunnels.
- Recently tunnels (Høyanger – Lånefjord Tunnel) were driven directly through and perpendicular to the mountain range between fjords or valleys instead of going around, reducing travel time and the need for avalanche galleries.

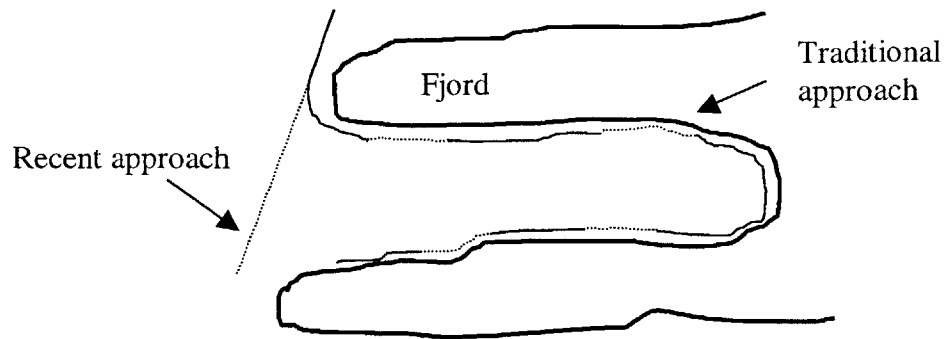


Figure 3.1: Routing of Tunnel

In both cases high anisotropic stresses caused rock bursts. The state of stress along the tunnel usually expected will be described here. The actual encountered conditions for the Høyanger Lånefjord tunnel will be described in section 3.1.1.1 and for the Heggura tunnel in Tafjord in section 3.1.1.2.

Figure 3.2 shows the direction of the principal stresses σ_1 and σ_3 for the traditional routing along the fjord. The depicted virgin state of stress with σ_2 is parallel to the tunnel axis does not change significantly along the tunnel alignment.

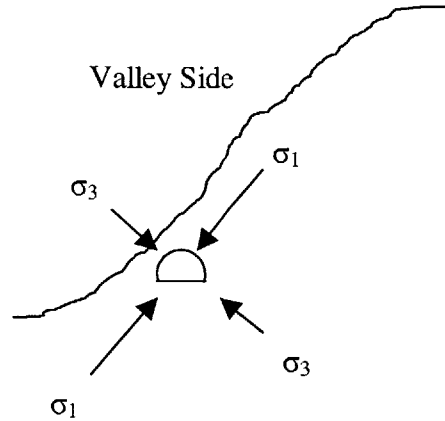


Figure 3.2: Principal Stresses, Cross Section, Traditional Routing

To understand the expected virgin stress environment along the tunnel for the recent routing, one needs to look not only at one cross section. Several cross sections along the tunnel alignment should be investigated, since magnitude and direction of principal stresses are expected to change. The principal stress σ_1 is approximately vertical and expected to increase with overburden. σ_2 is expected to be horizontal and perpendicular to the tunnel axis. σ_3 is expected to be perpendicular to the slope at the portals and horizontal and parallel to the tunnel axis in the center of the tunnel.

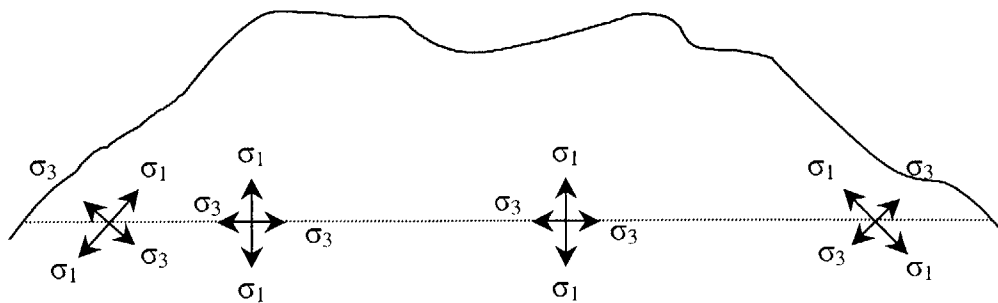
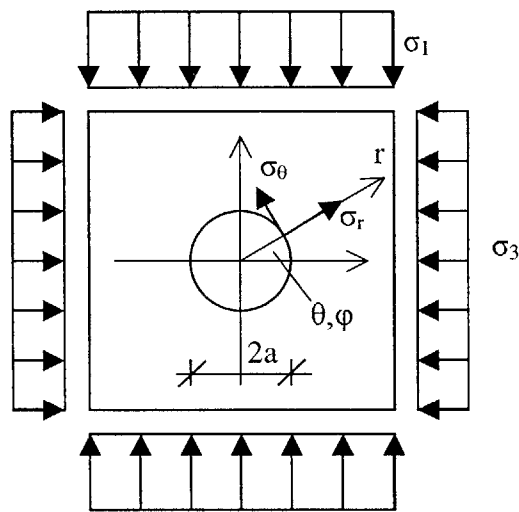


Figure 3.3: Expected Principal Stress Direction, Cross Section, Modern Routing

The change of the principal stresses was expected to increase rock burst activity along the “recent” tunnels as σ_1 increased with increasing overburden. Before the effect of the principal stress orientation can be shown it must be explained, why high anisotropic stresses are more likely to cause rock bursts than isotropic stresses of the same magnitude. This can be shown by a simple calculation with Kirsch’s (1898) (Equation 3.1) based on an infinite plate with a round hole in an isotropic homogeneous medium. This simplified model can also be used for three-dimensional excavations in the same stress field, since the two-dimensional solution is an upper boundary for the stresses (Sadowsky & Sternberg):



Definition of Symbols:

Stresses:

σ_r = radial stress

σ_θ = tangential stress

σ_1 = major principal stress

σ_3 = minor principal stress

Polar Coordinates:

a = radius of the hole

r = distance from origin

ϕ = direction of σ_r

θ = angle where $\sigma_\theta = \sigma_{\theta, \max}$

Figure 3.4: Round Hole in an Infinite Plate

The tangential stress perpendicular to direction “r” at distance r from the origin can be calculated as follows:

$$\sigma_{\theta} = 0.5\sigma_1 \left[\left(1 + \frac{\sigma_3}{\sigma_1} \right) \left(1 + \frac{a^2}{r^2} \right) + \left(1 - \frac{\sigma_3}{\sigma_1} \right) \left(1 + 3\frac{a^4}{r^4} \right) \cos(2\varphi) \right] \quad \text{Equation 3.1}$$

It has been found that the maximum tangential stress around holes in large plates always occurs on the inner boundaries of the holes. For the tangential stress at the inner boundary of the tunnel $a = r$ the equation 3.1 simplifies to:

$$\sigma_{\theta} = 0.5\sigma_1 \left[2 \left(1 + \frac{\sigma_3}{\sigma_1} \right) + \left(1 - \frac{\sigma_3}{\sigma_1} \right) 4 \cos(2\varphi) \right] \quad \text{Equation 3.2}$$

For constant σ_1 and σ_3 the maximum tangential stress $\sigma_{\theta, \max}$ at the angle $\varphi = \theta$ is determined by $\frac{d\sigma_{\theta}}{d\vartheta} = 0$.

$$\frac{d\sigma_{\theta}}{d\vartheta} = \sigma_1 \left[\left(1 - \frac{\sigma_3}{\sigma_1} \right) 4 \sin(2\vartheta) \right] = 4(\sigma_1 - \sigma_3) \sin(2\vartheta) = 0 \quad \text{Equation 3.3}$$

Since $4(\sigma_1 - \sigma_3) = \text{const}$, the maximum tangential stress can be found at $\theta = 0$. With $\cos(2\theta) = 1$ Eq. 3.2 simplifies to:

$$\sigma_{\theta, \max} = \sigma_1 \left(3 - \frac{\sigma_3}{\sigma_1} \right) = 3\sigma_1 - \sigma_3 \quad \text{Equation 3.4}$$

The maximum tangential stress around the tunnel is therefore $\sigma_{\theta, \max} = 3\sigma_1 - \sigma_3$. For standard cases with vertical σ_1 this implies, that $\sigma_{\theta, \max}$ increases rapidly with increasing overburden and decreases with increasing horizontal stress.

To make a comparison of the Heggura tunnel in Tafjord and the Høyanger – Lånefjord Tunnel easier, key project data are shown in Table 3.1:

	The Høyanger Lånefjord Tunnel “recent routing”	The Heggura tunnel in Tafjord “traditional routing”
Year	1976-1982	1980
Length, l	7522 m	5226 m
Cross Section, A	semicircular, 50 m ²	semicircular, 39 m ²
Max. Overburden, z	up to 1100 m	700 m
Uniaxial Compressive Strength σ_c of rock	60 to 200 MPa	100 to 250 MPa
Young’s Modulus, E	30000 to 50000 MPa	10000 to 30000 MPa
Major Principal Stress σ_1	34 MPa	25 MPa
Minor Principal Stress σ_3	9 MPa	?
Max. Tangential Stress $\sigma_{t, \max}$	93 MPa	?
Location of rock bursts	Roof*	Lower sidewalls on the mountain side and upper sidewall on the valley side
Construction Method	Drill and blast	Drill and blast
Driving rate		
Costs		
*) See Figure 3.8		

Table 3.1: Project Data

3.1.1.1 Høyanger Lånefjord Tunnel

The Høyanger Lånefjord Tunnel is in the west of Norway, about 60 km northeast of the city of Bergen (Figure 3.5, Figure 3.6)



Figure 3.5: Southwest of Norway

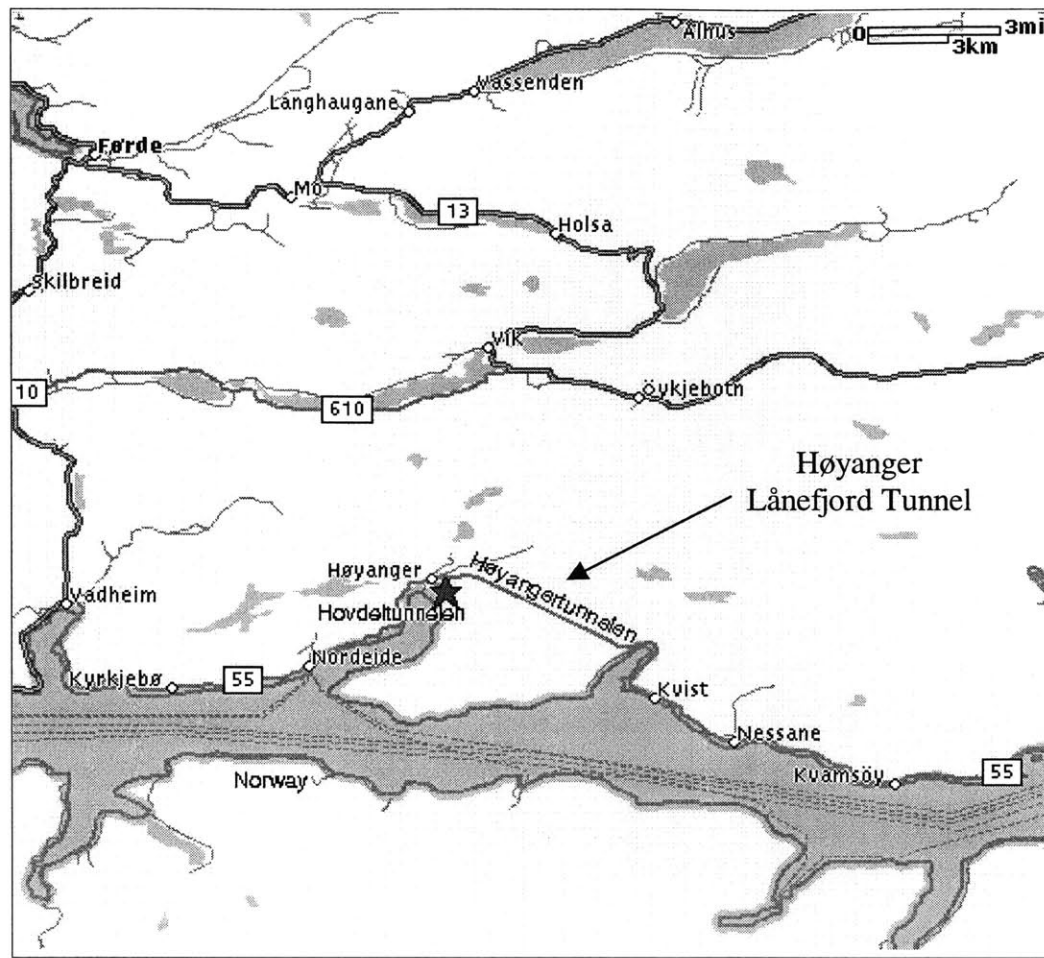


Figure 3.6: Location of the Høyanger Lånefjord Tunnel

It was driven from both portals by conventional drilling and blasting. One portal is at Høyanger and the other one at Lånefjord. They will be referred to as the Høyanger side and Lånefjord side. At the Høyanger side construction started in November 1976 and one year later at the Lånefjord side. The maximum overburden of 1100 m is approximately in the middle of the tunnel. The rock mass consists mainly of different types of gneiss and has relatively few fracture zones. Lenses of amphibolite occur frequently and rarely a few layers of quartzite. The uniaxial compressive strength σ_c ranges from $\sigma_c = 60$ MPa to $\sigma_c = 200$ MPa and the Youngs' modulus from $E = 30000$ MPa to $E = 50000$ MPa.

As explained for the “recent routing” rock bursts were mainly anticipated far away from the portals at maximum overburden due to high vertical stresses (Figure 3.3). The location on the circumference of the tunnel was predicted as shown in Figure 3.7 in the sidewalls of the tunnels, since the maximum tangential stress occurs on the sides for vertical σ_1 (Equation 3.3).

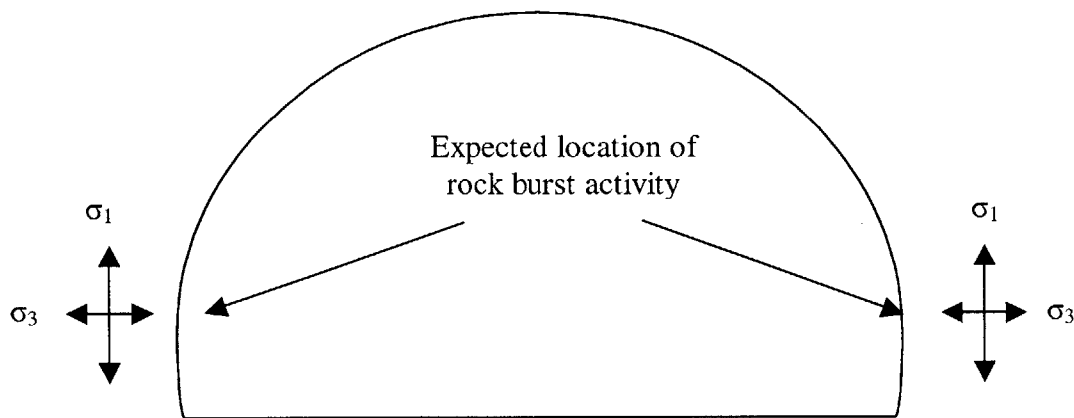


Figure 3.7: Expected location of rock burst activity and direction of principal stresses

How the encountered situation differed from the anticipated state of stress and the effect on the rock burst pattern will be explained on the next pages:

3.1.1.1.1 Høyanger Side

After construction started and it became obvious that the predictions were wrong, an investigation was carried out to clarify the situation. The main observations were:

- Rock bursts were encountered much earlier than predicted at moderate overburden about 200 m away from the portal at the Høyanger side.
- Rock burst occurred in the roof of the tunnel instead of the sides. (Figure 3.8)

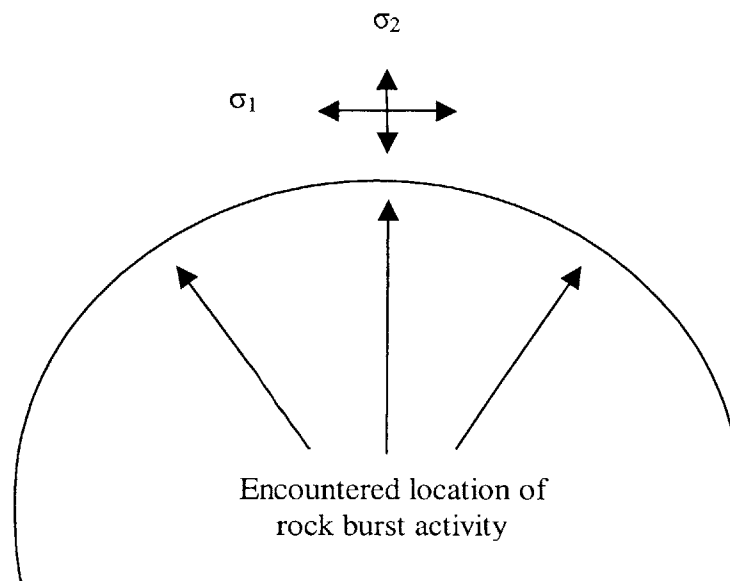


Figure 3.8: Encountered location of rock burst activity and direction of principal stresses 200m from the face

It was concluded from this observation that the major principal stress σ_1 must be horizontal. Triaxial in-situ stress measurements confirmed this. More precisely it was found, that σ_1 is horizontal and perpendicular to the tunnel axis and parallel to the mountain range, which is parallel to the fjord (Figure 3.9, Figure 3.10).

To understand the virgin stress environment along the tunnel and make predictions for rock burst activity along the tunnel, one needs to look at both the longitudinal cross section (Figure 3.9) and the plan view (Figure 3.10) of the entire tunnel alignment. This shows, that the major principal stress σ_1 is horizontal and approximately perpendicular to the tunnel axis along the entire tunnel. Close to the portal on the Høyanger side σ_2 is in the tunnel axis and vertical in the rest of the tunnel. σ_3 is vertical close to the portal on the Høyanger side and in the tunnel axis in the rest of the tunnel.

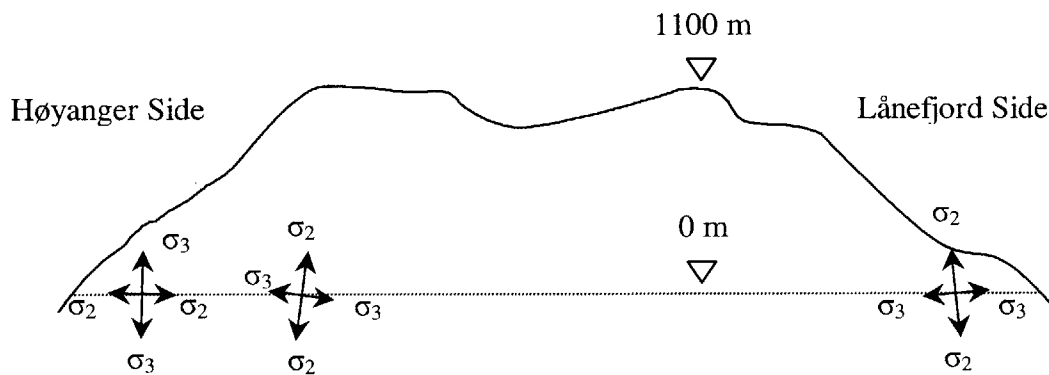


Figure 3.9: Principal Stress Directions from Measurements, Cross Section, Høyanger Lånefjord Tunnel

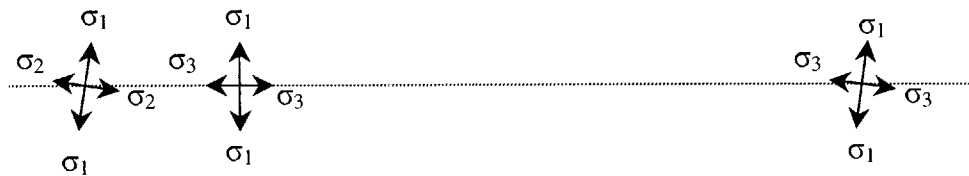


Figure 3.10: Principal Stress Directions from Measurements, Plan View, Høyanger Lånefjord Tunnel

Even though the tunnel has a semicircular shape, the assumption of the simple elastic model described above, the circular hole in an infinite plate, is accurate enough to illustrate the state of stress and the effect on rock burst around the sides and the top of the tunnel. The rationale of the illustration is not to calculate precise values, but to make use of the observational method in order to change design, construction method and support.

To illustrate the stress change and its effect, three cross sections at different overburden will be examined:

- Cross section A, 200m from the portal on the Høyanger Side at moderate overburden,
- Cross section B, at 900m overburden
- Cross section C, at the maximum overburden of 1100m

From the in situ measurements the state of stress was known in cross section A. The vertical stress $\sigma_v = \gamma * h$ with $\gamma = 26.5 \text{ kN/m}^3$ can be determined in cross section B and C with good accuracy from the overburden (Table 3.2). Predictions were made based on constant σ_1 and varying σ_v . The rationale is, that the horizontal stress $\sigma_1 = \sigma_h$ is constant along the tunnel except in the near surface zones, which are influenced by erosion and local topography. From

$$\sigma_{\theta} = 0.5\sigma_1 \left[2 \left(1 + \frac{\sigma_3}{\sigma_1} \right) + \left(1 - \frac{\sigma_3}{\sigma_1} \right) 4 \cos(2\varphi) \right] \quad \text{Equation 3.2}$$

one can obtain the tangential stress in the roof for $\varphi = 0$ and in the sides for $\varphi = \pi/2$. Therefore the tangential stress in the roof is $\sigma_{\theta, \max} = 3\sigma_h - \sigma_v$ (Eq. 3.4) and $\sigma_{\theta, \min} = \sigma_h - 3\sigma_v$ in the sides. The tangential stresses in the roof and the sides of the tunnel at Cross sections A, B and C are shown in Table 3.2 and Figure 3.11.

Cross Section	A	B	C
Overburden [m]		900	1100
Stresses in [MPa]			
$\sigma_1 = \sigma_h$	34	34	34
$\sigma_v = \sigma_{2,3} = \gamma \cdot h$	9 ⁽¹⁾	24 ⁽²⁾	29 ⁽²⁾
$\sigma_{\theta, \text{roof}}$	93	78	73
$\sigma_{\theta, \text{wall}}$	-7	38	54
⁽¹⁾ from in situ measurement			
⁽²⁾ prediction with $\sigma_{2,3} = \sigma_v = \gamma \cdot h$			

Table 3.2: σ_{θ} in the roof and the wall of the tunnel

For better illustration, Figure 3.11 shows the tangential stress around the tunnel for all 3 cross sections. The different stress conditions in the three cross sections are normalized to the highest encountered tangential stress, so that one unit length distance in the diagram equals $\sigma_{\theta} = 93$ MPa (cross section A). The diagram shows clearly that max σ_{θ} is predicted to decrease in the crown from cross section A to C as the overburden increases. The second term in $\sigma_{\theta, \text{max}} = 3\sigma_1 - \sigma_3$ (Eq. 3.4) is often underestimated, but accountable for this decrease. It is also worthwhile noting, that in cross section A the walls of the tunnel were not in compression, but in tension because σ_v is much lower than σ_h .

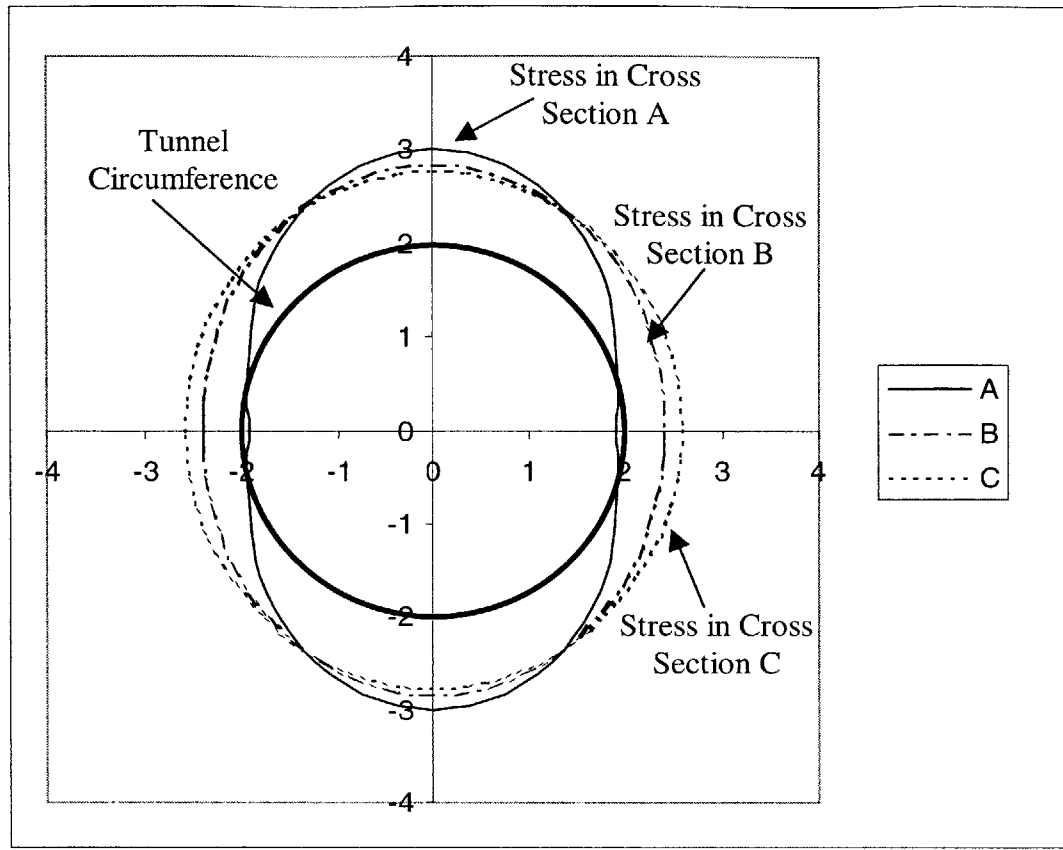


Figure 3.11: Tangential Stress Distribution around the Tunnel

From the known stresses and the observed rock burst activity in cross section A and the predicted stresses in cross sections B and C, rock burst activity was predicted as follows:

- As the overburden increases rock burst activity in the roof will be less prominent. However the tangential stresses ($\sigma_{\theta, \text{roof}} \geq 73 \text{ MPa}$) will still be close enough to the rock's uniaxial compressive strength $\sigma_c = 60 \text{ to } 200 \text{ MPa}$ to cause rock bursts.
- Under the highest overburden (cross section C) stresses in the wall will increase to a level that will lead to rock burst activity in the sidewalls.

During the construction process in the following years these predictions turned out to be quite accurate. The predicted principal stress conditions were confirmed by measurements and rock bursts occurred mainly in the roof and at high overburden in the roof and in the sidewalls.

Some deviations in the predicted pattern were noticeable:

- Spalling and rock bursts in the face, which occurred from 450 m to hole through and are described as “quite heavy”. Why this was not predicted is unclear. The direction of the major principal stress σ_1 lies in the plane of the face. These are the most unfavorable conditions that can be encountered.
- Erratic rock burst pattern and intensity occurred, when crossing fracture zones and a few lenses of amphibolite. Fracture zones and lenses of different rocks cause local stress concentrations, which are responsible for this erratic behavior.

3.1.1.1.2 Lånefjord Side

Since construction on the Lånefjord side started about a year later experience and predictions from the Høyanger side were used. The predictions proved to be reasonably accurate, but did not take into account different geologic conditions. In the first 500 m to 600 m no rock bursts occurred, since the rock was jointed and thus prevented development of high tangential stresses. After this point the rock was intact and considerable rock bursting from the roof and the face occurred and increased until 2000 m from the portal. After this point the intensity decreased until hole through. Accountable for the intensive

rock bursts at high overburden were local stress concentrations due to frequent changes between intact gneiss and schistose amphibolite.

3.1.1.1.3 Prevention, Support System, Construction Procedure

Prevention and Support System

To control the spalling and rock burst problems the affected areas were systematically bolted (Figure 3.12)

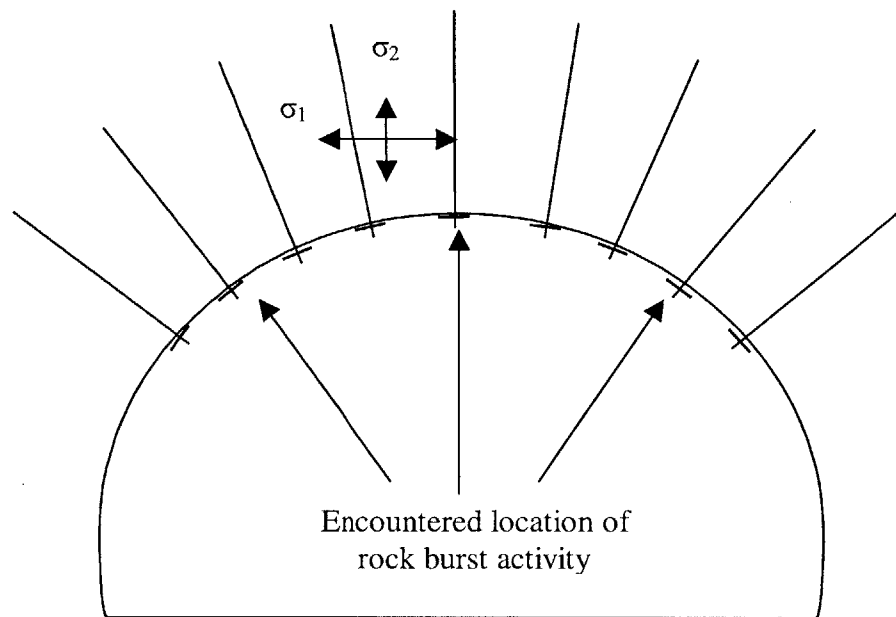


Figure 3.12: Bolting Scheme for Rock Burst in the Roof

The bolts used were \varnothing 20mm rebar of 2.40 m length. They were point anchored with resin cartridges and the triangular roof plates had a size of 25 cm². It is reported, that to observe the strength of the bolts, pull out tests were performed. These demonstrated, that bolts were torn off at a force of about 180 kN (~ 55 kN/cm²). It seems though that the determination

of the mode of failure was the objective of the investigation, since this failure load could have been determined reasonably accurately by the materials properties. However only a few bolts were reported to have been torn off by rock bursts. Depending on rock burst intensity, 35 to 70 rock bolts were used per 4m round (9 to 18 per m tunnel). Wire mesh was installed in the roof through very intensive bursting zones. In the worst zones the face was bolted with 1 m bolts and sometimes wire mesh was also applied to the face.

Construction Procedure

To install the bolts a two boom pneumatic drill with a working platform was used on the Lånefjord side. At the Høyanger side drilling was carried out with hand held jacklegs, which were operated from a hydraulic platform.

Rock burst intensity influenced the construction procedure. The contractor chose two different round cycles according to the intensity:

- In sections with heavy rock bursts half of a round was mucked out first. Next scaling and rock bolting was carried out as far as possible. Then mucking was completed and scaling and bolting was finished up to the face. After that, if necessary, wire mesh was installed and/or the face was bolted.
- In sections with moderate spalling, scaling of the entire round was carried out before mucking. After scaling the bolting for the complete round was completed.

3.1.1.1.4 Impact of rock bursts on construction time and cost

Before the project started, the driving rate with 10 shifts was estimated to be about 10 rounds per week (40m/week). In problematic zones this rate was sometimes as low as 5 rounds per week (20m/week). The cost per m of blasted tunnel on the Høyanger side as a function of the driving rate is shown in Figure 3.13.

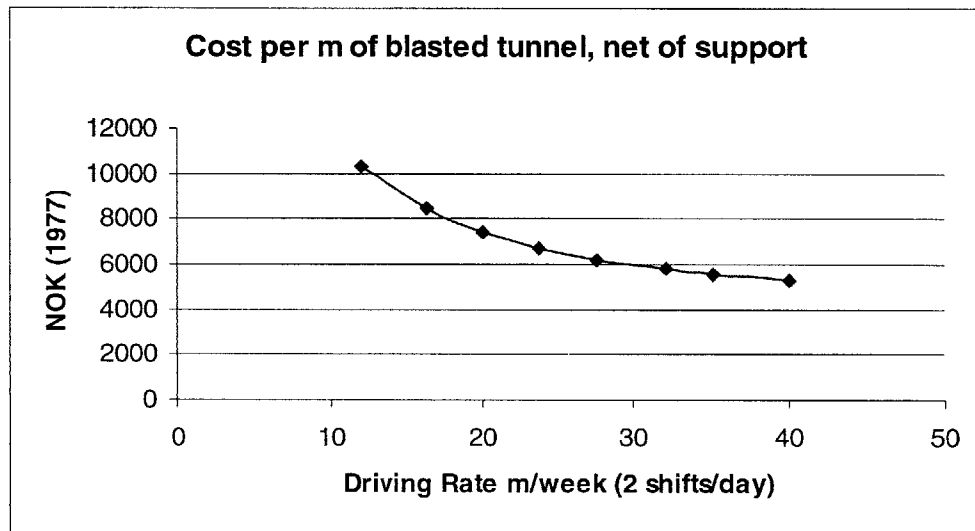


Figure 3.13: Cost as a function of driving rate

With an anticipated driving rate of 40 m/week the cost per m of blasted tunnel was NOK 5308 (Norwegian Kroner, 1977). For the problematic zones the cost the driving rate dropped and cost increased up to NOK 10307 (1977). Additional costs for the bolts and the wire mesh of NOK 85 (1977) per bolt and NOK 40 per m of wire mesh have to be added to the cost in Figure 3.13. The average rate in 1977 was 22m/week. The cost for this rate are NOK 7680 per m (NOK 7000 per m plus NOK 680 for ~ 8 bolts per m). As a result, rock stress problems caused a 44 % increase in cost over the anticipated driving rate of 40 m/week. No specific data is available for the Lånefjord side. The extra cost due to rock

burst problems are in the same order (NOK 2800 per m) as for the Høyanger side (NOK 2370 per m).

3.1.1.2 Heggura Tunnel in Tafjord

The Heggura Tunnel in Tafjord is located in the west of Norway, about 250 km southeast of the city of Trondheim (Figure 3.14).

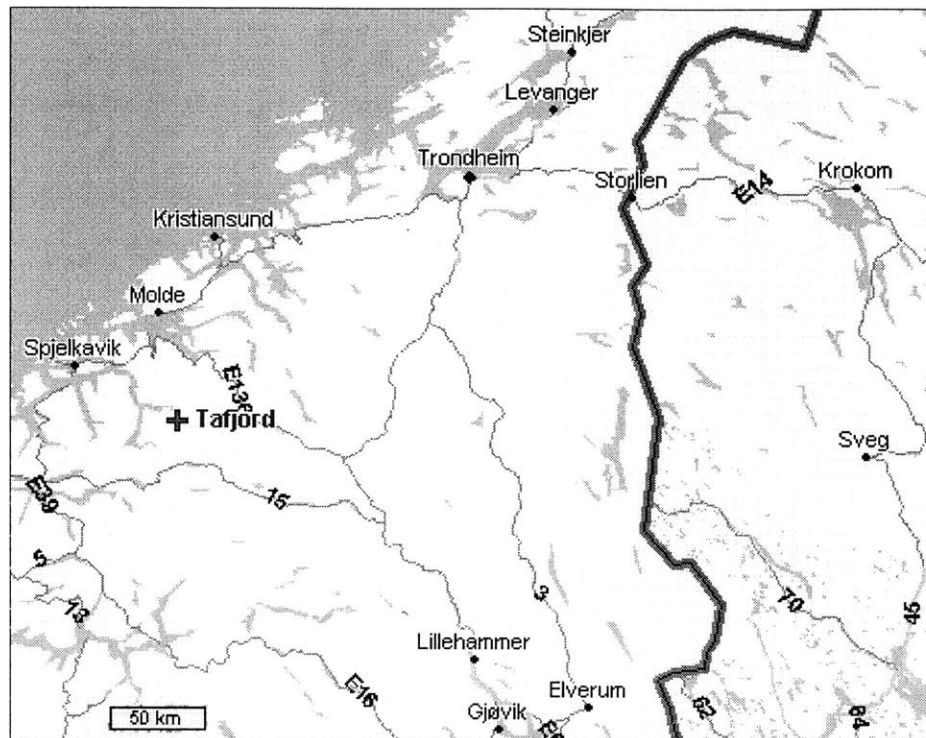


Figure 3.14: West of Norway

In this part of Norway tunnels along the up to 1500m high and 45° or steeper shores of fjords are very common. The main reasons for locating roads in tunnels are the very frequent avalanches and rock falls in the steep mountains in the area. The Heggura Tunnel in Tafjord falls in the “traditional” routing category as depicted in Figure 3.1 and Figure 3.2. The village of Tafjord (Figure 3.15) only has about 200 inhabitants (1980). Before the tunnel was built a ferry served the village 4 to 5 times a day. The reason for building the

tunnel was that the headquarters of five hydro power plants producing 813 GWh a year are located in Tafjord.

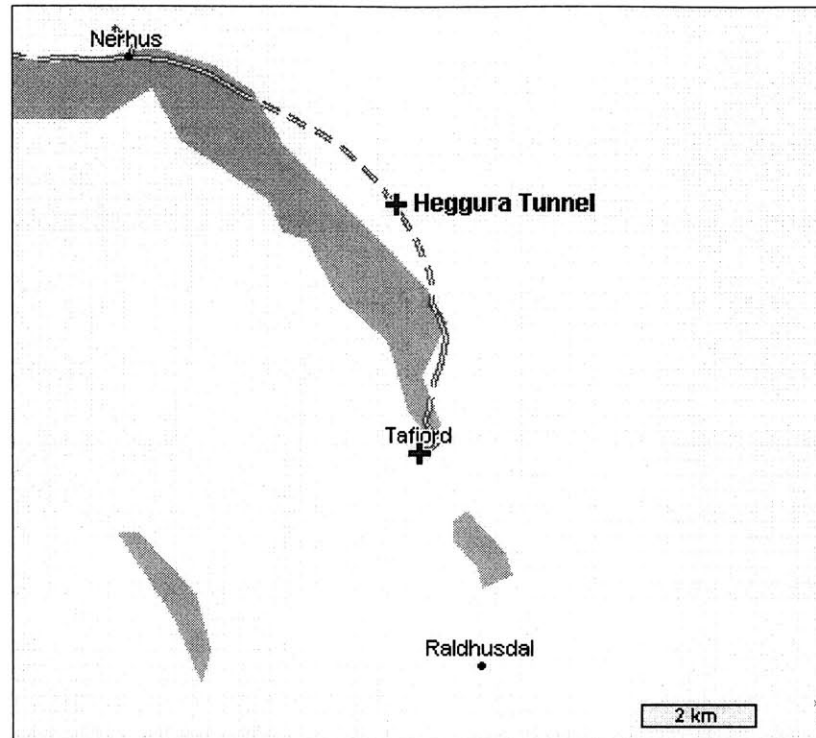


Figure 3.15: Tafjord and the Heggura Tunnel

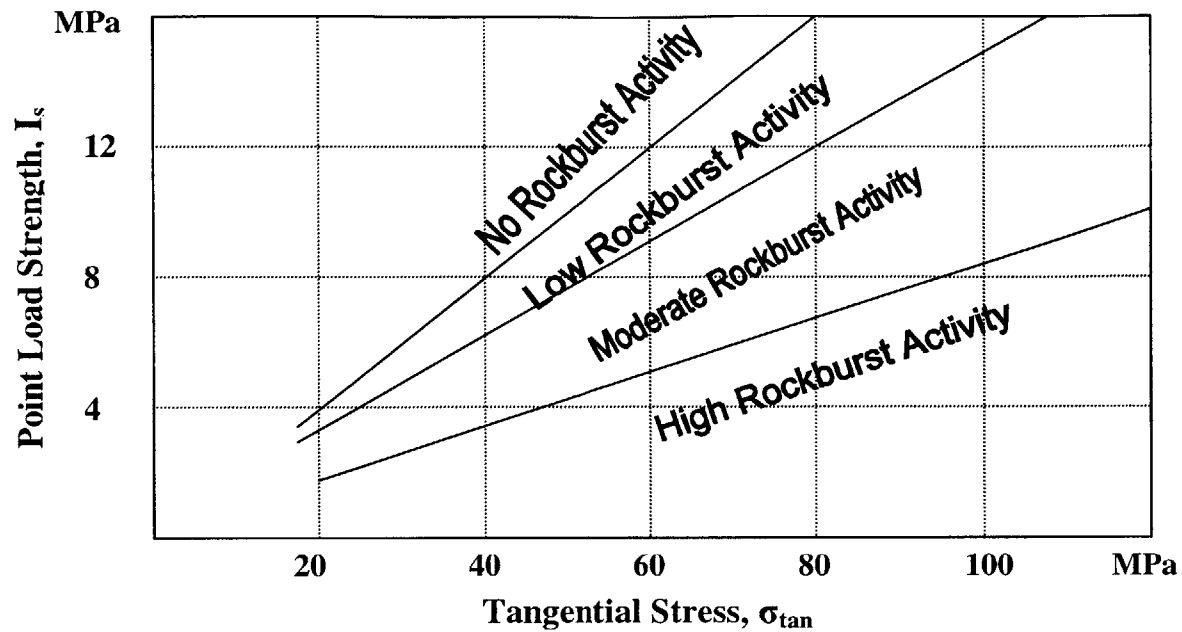
The mountain range in Tafjord is parallel to the fjord and mountains rise steeply up to 1500m above sea level. Therefore a tunnel along the fjord would be subject to high anisotropic stresses. Since many tunnels have been built along Norwegian fjords significant experience in these conditions was available. An old rule of thumb (Selmer-Olsen, 1965) states that if a mountain along the fjord is higher than 500m and has a slope exceeding 25° , rock stress induced stability problems are likely to occur in tunnels at the base of the mountain. Unsurprisingly the dominating type of stability problem encountered in the tunnels of this region is rock bursting.

Precambrian gneisses dominate the Tafjord area. Among the various rock types two gneiss types are clearly dominating:

- A gray, banded, medium to coarse-grained gneiss with high quartz and feldspar contents.
- Dark, fine-grained gneiss rich in mica.

The relatively constant foliation of the rocks is east west and the dip varies due to bending of the rock. From the experience gained in the pressure tunnels of the nearby power plants in the same geological formation it was assumed, that the precambrian gneisses would be of very good rock quality. Preinvestigations indicated, that no major fault zones were to be expected. These positive predictions were supported by the experience in the of the nearby “Grytten Kraftwerk” power plant. Here the 25 km long diversion and headrace tunnels only had to be supported by a few hundreds rock bolts. However, the higher elevation and lower overburden of these tunnels should be kept in mind when applying experience to a new project with different boundary conditions.

The expected dominating stability problem during construction was, like for the Høyanger Lånefjord Tunnel, rock bursting resulting from high anisotropic stresses. In the design of the tunnel, rock burst activity was classified into four classes according to tangential stress and point load strength (Figure 3.16). A classification into four rock groups was also done. (Table 3.3). Unfortunately rapidly changing gneisses along the tunnel made mapping and classification difficult.



Rock Burst Class	Description
0	No Rock Bursting No stability problems caused by rock stresses. No noises from the rock.
1	Low Rock Bursting Activity Some tendencies to cracking and loosening of rock. Light noises from the rock.
2	Moderate Rock Bursting Activity Considerable slabbing and loosening of rock. Tendencies to develop a deformed periphery with time. Strong cracking noises from the rock.
3	High Rock Bursting Activity Severe rock falls from the roof and walls immediately after blasting. Slabs pop from the floor, or the floor may heave. Considerable over-breaks and deforming of the periphery. Rock noises of gun shot strength may be heard.

Figure 3.16: Classification According to Russenes, 1974

Rock Group	A	B	C	D
Typical rocks	Augengneiss Garnetgneiss	Banded gneiss Amphibolite	Biotite gneiss	Micarich gneiss
Content of mica and amphibolite	15 %	25%*	30%	40%
Point load strength [MPa]	10	7	5	3
Uniaxial compr. Strength [MPa]	200	150	100	50
Youngs Modulus [MPa]	40	30	20	10
* Not valid for amphibolites				

Table 3.3: Typical Rocks in the Heggura Tunnel

To reduce tangential stresses the center line of the tunnel was placed as deep into the hill as economically possible. To account for the anisotropic stresses, the contract contained two alternative cross sections (Figure 3.17), in case rock bursting got severe.

The tunnel was built using the conventional drill blast method from both sides. Rock stress problems were encountered during about 3000 m of the 5226 m long tunnel. Despite the problems hole through was achieved within only 13 months, giving an advance rate of 53m per tunnel face per week. Work was done in eleven 10 hour shifts per week.

Temporary support

Rock bolts were mostly installed on the valley side half of the tunnel roof. Out of the 15500 2.50 m long resin grout rock bolts only a few hundred were installed in the mountain side of the roof. The bolts were installed close to the tunnel face, generally supporting the last round, to prevent induced spalling. The bolts were installed with 150 mm round disks and were generally not tensioned. In areas with high rock bursting activity, fragments of rock loosened under the disks, especially for bolts installed close to the face, where the next blast gave an additional shock to the rock mass. Such bolts became inactive and were reactivated by retightening the nuts.

Shotcrete was used as soon as rock bursting problems were encountered to protect men and equipment against rock fragments popping out between the rock bolts. After a few rounds the contractor and the owner agreed on the use of steel fiber reinforced shotcrete. The reasoning for using the 50 % more expensive steel fiber reinforced shotcrete was its better strength and deformation properties and that the costs for the final support could be reduced.

The normal tunneling round in the 3 km rock burst section was as follows:

1. Drilling
2. Charging and blasting
3. Scaling from a hydraulically operated basket combined with marking for rock bolts
4. Mucking and hauling
5. Installing of rock bolts (additional scaling if necessary)

6. First 5 cm layer of shotcrete applied to last but one round
7. Second 5 cm layer of shotcrete applied to last but two rounds

The first attempt of applying steel fiber reinforced shotcrete as a first support in high rock burst activity zones was unsuccessful. The rock spalled, before the shotcrete was able to set and the contractor changed to the round pattern described above. For safety reasons, the shotcrete was installed by a hydraulic robot and the rock bolts had to be installed from an already supported part of the tunnel. Figure 3.17 shows the asymmetric alignment of shotcrete and rock bolts. This was economical since Norwegian tunnels are built without liner. At the second point of tangency of the major principal stress, the mountain side corner of the floor rock bursting was frequently observed, but since it was not a safety threat no support was placed. In the most severe zones of rock bursting the whole face had to be covered with reinforcement meshes, which were bolted to the face before the next round was drilled.

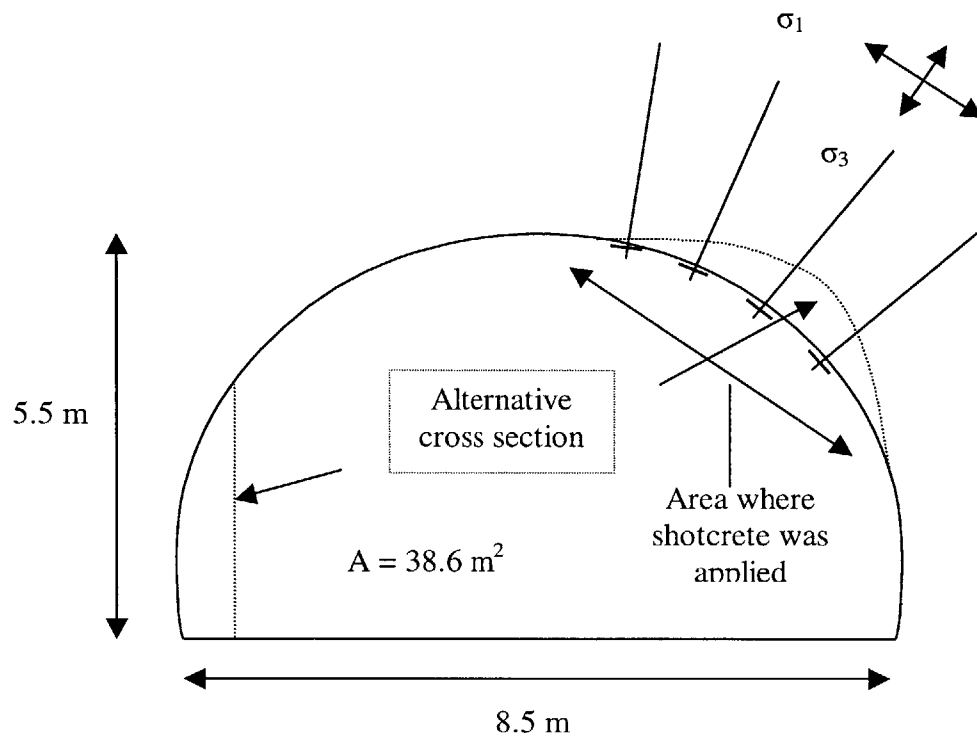


Figure 3.17: Alternative Cross Section

As far as the classification of areas into rock burst classes was possible, the number of bolts needed correlated directly with a higher rock burst class (Figure 3.18). In the worst zones as many as 80 rock bolts were needed per round.

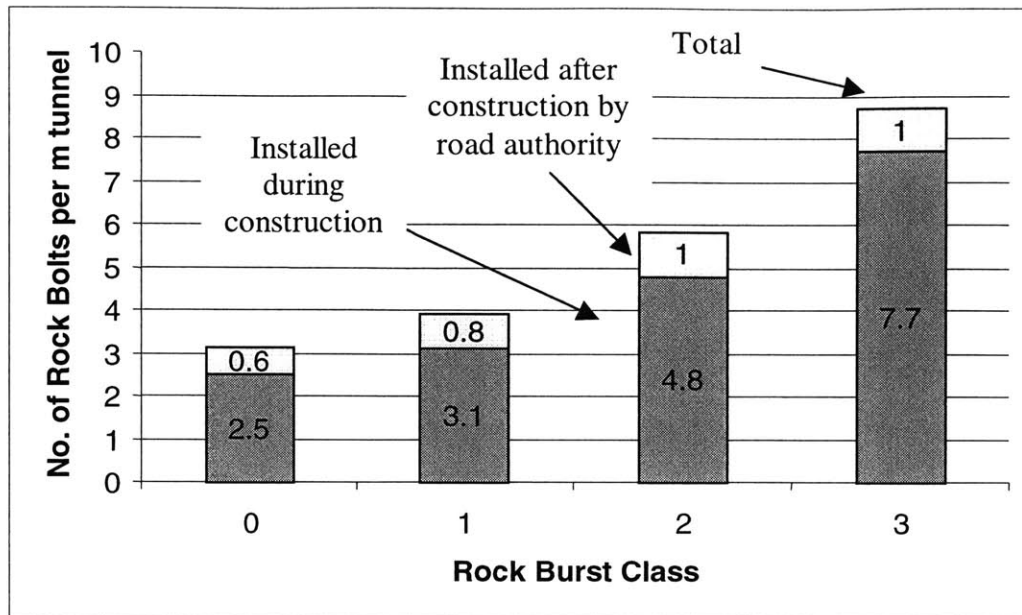


Figure 3.18: Number of Rock Bolts Needed as a Function of Rock Burst Class

Final Support

Since traffic is very low no ventilation and only reduced lighting is installed in many Norwegian tunnels. To reduce hazards due to freezing leakage water in the winter, panels that divert the water to the side drains were installed. During final scaling few new slabs loosened, but were taken down or bolted. No additional shotcrete was applied. In areas where the shotcrete from the temporary support had cracked or loosened, it was taken down or bolted. For this final work 5250 rock bolts with lengths from 2.0 m to 2.4 m were used. Adding these to the previously installed bolts the total amount of bolts for the 5260 m long tunnel was 21000, or 3.9 bolts per m. This is significantly less than the 10 rock bolts per m in the Hoyanger tunnel where no shotcrete had been used.

In summary the observations and lessons learned from the Heggura tunnel are:

- Rock bursting activity increases when the tangential stress increases. This effect was fairly well pronounced for rocks in rock group D (Table 3.3). For better rocks the effect is, however, less well pronounced and no effect can be seen for rock group A.
- Rock bursting activity in rock group A is stronger than in D. Strong rocks can store more strain energy and therefore failure will be more violent. For the weaker rocks failure was at a lower stress level and a yielding of the rock mass was seen.
- Rock cores extracted from the tunnel walls showed that for the stronger rock in group A and B, secondary cracking occurs close to the tunnel within 20 to 30 cm from the surface. For the weaker rocks, in group C and D, secondary cracking due to high stress was observed as deep as 1 m from the surface. Thus the weaker rocks will give a deeper zone of stress reduced rock masses around the tunnel and therefore less intensity of rock bursting. The weaker rocks may be expected to produce give higher deformations, however no deformation measurements were carried out at the Heggura tunnel.
- In no areas borings encountered secondary cracking deeper than 1 m from the tunnel surface. This implies that for pre-rock burst stability problems, rock bolts can be shorter than they are used today. For stability problems caused by primary joints, however longer bolts may be needed.

- Rock bolts installed close to the face had a tendency to become inefficient in areas with high rock burst activity. This effect was stronger for rock bolts installed at an oblique angle.
- Compared to the use of conventional shotcrete, the use of steel fiber reinforced shotcrete made it possible for the contractor to keep a high speed of tunneling even through zones of heavy rock bursting.
- The use of shotcrete strongly reduced the necessary scaling in the tunnel. Costs for so-called extra scaling had been increasing rapidly before shotcreting was started.
- 90% of the steel fiber reinforced shotcrete was still intact after two years. This implies that for tunnels on secondary roads it can be used as a final support.
- The use of steel fiber reinforced shotcrete reduced the feeling of being unsafe among tunnelers. They could easily see the effect of using it, and very soon they felt confident with this support.

3.1.2 Kan Etsu Tunnel, Japan

The Kan Etsu Tunnel is part of the Kan Etsu Expressway, which links the Japan Pacific Highways in the east with the coast of the Japanese Sea in the west. It connects the Tokyo region with the cities of Niigata and Naga-Oku (Figure 3.19).

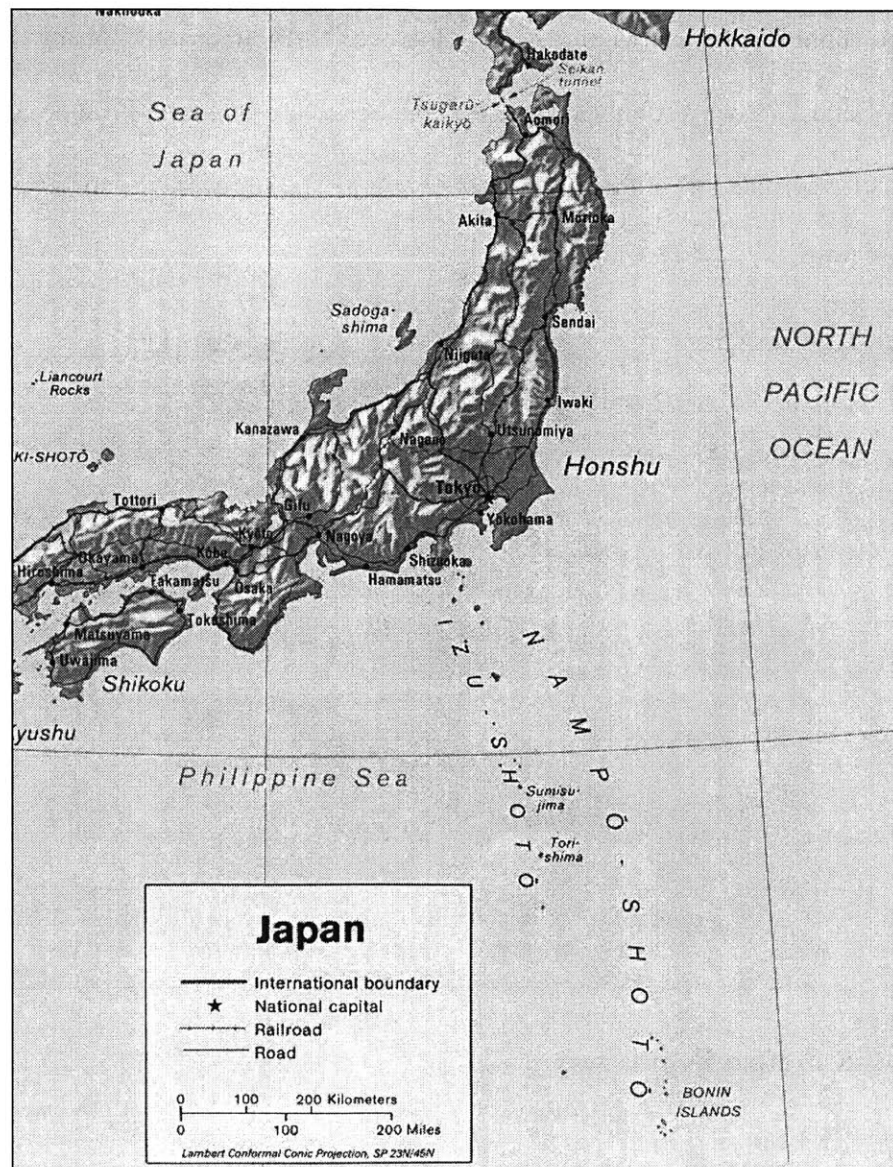


Figure 3.19: Japan, Island of Honshu

Numerous tourists from the urban Tokyo region travel to the Japanese Sea, because of its beaches and scenic views. Before 1985 the expressway ran across a mountain range with snaking roads and passes up to 1800m altitude (Figure 3.20). To reduce traffic jams due to lack of capacity and closings of the passes due to bad weather conditions the first Kan Etsu tunnel was opened in 1985. It lies at 1100m altitude and with 10925m length, it is the longest road tunnel in Japan (Figure 3.20). However traffic increased faster than expected and the two lane two-way tunnel's capacity was exceeded quickly. To increase capacity a second tunnel was planned. After completion each of the tunnels was to be used as a two lane one way tunnel.

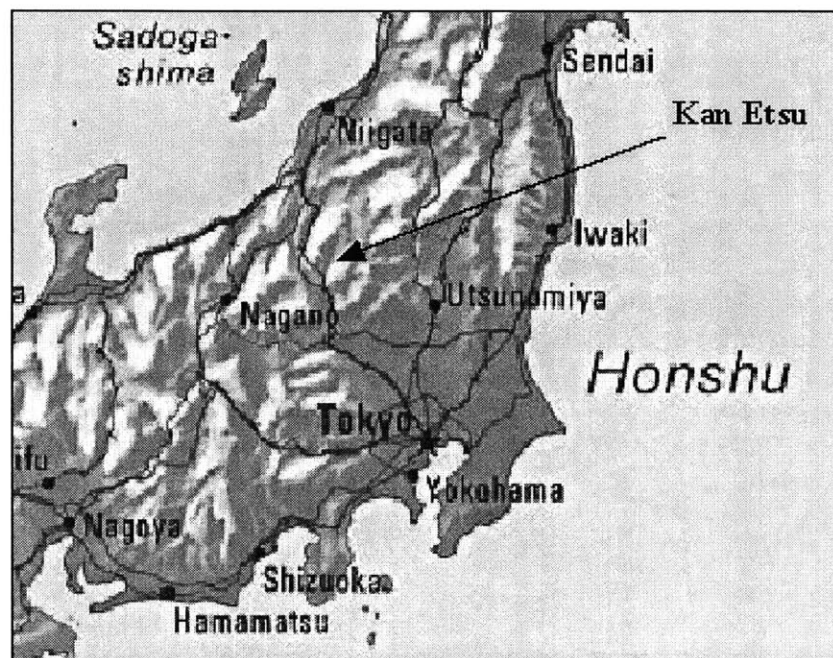


Figure 3.20: Kan Etsu Expressway

Both Kan Etsu Tunnels were driven through hard volcanic rock in. The geologic formation consists mostly of quartz, diorite and hornfels. The mountain forming process is still active, which is very common for Japan and results in high residual stresses. The residual

stresses in the area are described as “complex, difficult to predict and of large magnitude”. It is likely that the “complex stress field” at Kan Etsu is caused by the combination of tectonic stresses and the mountain forming process.

After bad experience with rock burst problems during construction of the first Kan Etsu Tunnel it was clear that one would face similar challenges when building the second tunnel. Unfortunately no records of the design and construction of the first tunnel could be found in the literature.

Construction of the second Kan Etsu Tunnel started in fall of 1986. Excavation was carried out using the drill and blast method. Work was executed by two Japanese joint ventures:

Taisei-Nishimatsu-Sato, working from the northern portal

Ohbayashi-Shimizu-Tobishima, working from the southern portal

During the first years of construction shotcrete and pattern bolting was applied as temporary support under normal conditions. 3m long steel rebars with CEMBOLT Portland cement cartridges (manufacturer: OPTIROC, Sweden) were used as rock bolts. In sections with hard, brittle rock, with few fractures, rock bursts took place around the tunnel perimeter and in the face. The rock bursts experienced at Kan Etsu are commonly referred to as strain bursts. Strain burst occur in the immediate vicinity of the excavation. Even without knowing much about the state of stress in the rock, one can deduce from this observation that the rearranged stress field has the typical stress concentration around the sides of the excavation parallel to the initial major principal stress (Figure 3.21).

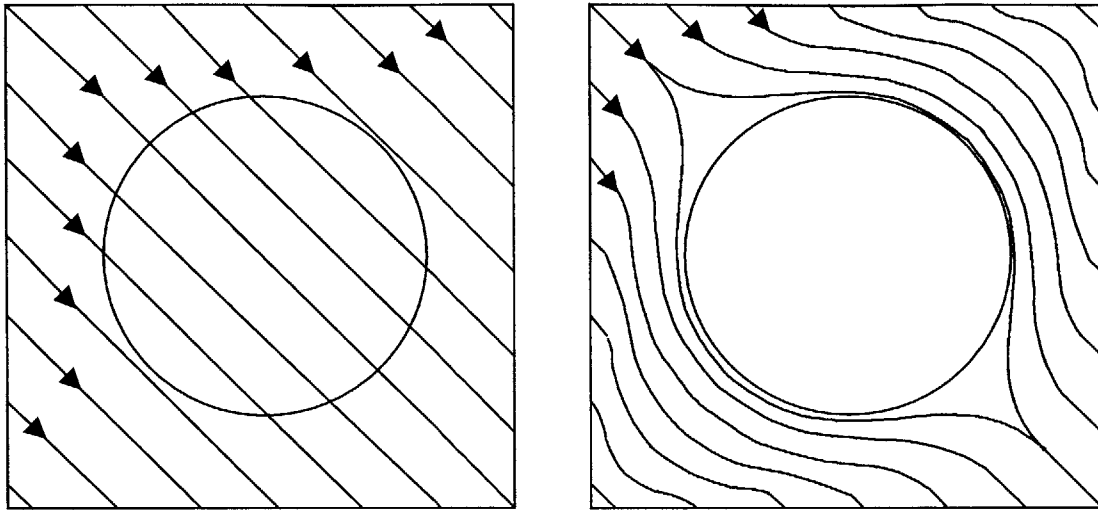


Figure 3.21: Stress Field Before and After Excavation

Rock bursts in the face and around the tunnel caused significant delays. The three main reasons were:

- At the face and around the tunnel: Waiting periods to ensure workmen safety. The contractor had to wait until rock bursts halted and stresses dissipated.
- At the face: Drill hole spalling. Rock bursts occurred inside drill holes, which hindered drilling and loading of blasting holes, because of debris in the hole (Figure 3.22). As rock bursts in the tunnel, drillhole spalling is mainly observed in highly stressed brittle rock.

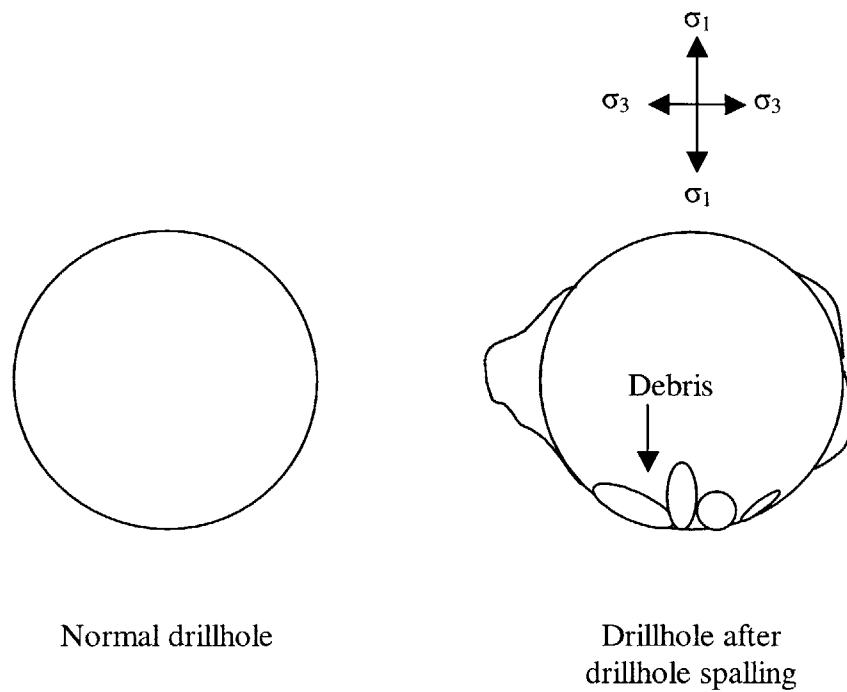


Figure 3.22: Drillhole Spalling

- In the roof: Drillholes with steps. Bolting the roof was slowed down because of steps in the drill holes made inserting the bolts difficult (Figure 3.23). The steps are caused by fractures that occur after the hole has been drilled. Rockbursting activity does not have to be high to cause these steps, since it is fracturing that does not result in ejection of rock, which is the main cause of the steps.

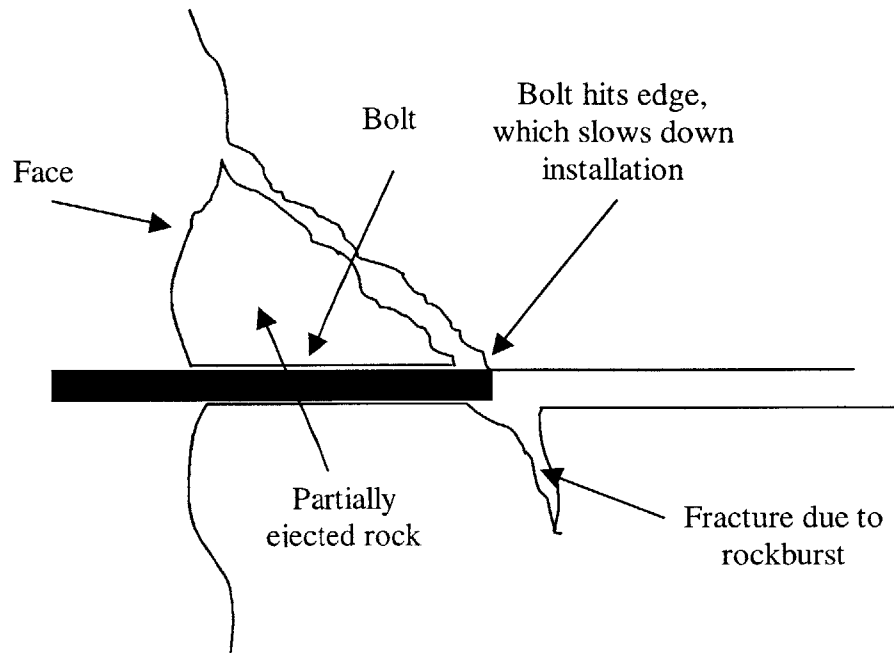


Figure 3.23: Drillholes with Steps

Under stress and rock conditions similar to the prevailing conditions at Kan Etsu, destress blasting can be used to precondition the rock and avoid rock bursts. Destress blasting simply creates fractures in the high stress area and induces movements, which relax stresses. One disadvantage of distress blasting however is the fine-tuning of a satisfactory blasting arrangement. It takes too much time for tunneling project where delays are very costly.

To avoid schedule delays due to rock bursts the joint venture Taisei-Nishimatsu-Sato evaluated different rock stabilization systems, which could prevent extensive rock bursts and avoid interruption of the tunneling process:

- First fiberglass bolts with fast hardening resin were used in the face (Figure 3.24 (i)). The result was disappointing: After blasting nothing of the bolt remained in the hole to stabilize the rock and rock bursting continued (Figure 3.24 (ii)).
- As second alternative steel rebars with fast hardening resin were used in the face (Figure 3.24 (i)). Again results were unsatisfactory: Nothing of the bolt remained in the hole to stabilize the rock after blasting and rock bursting continued (Figure 3.24 (ii)).

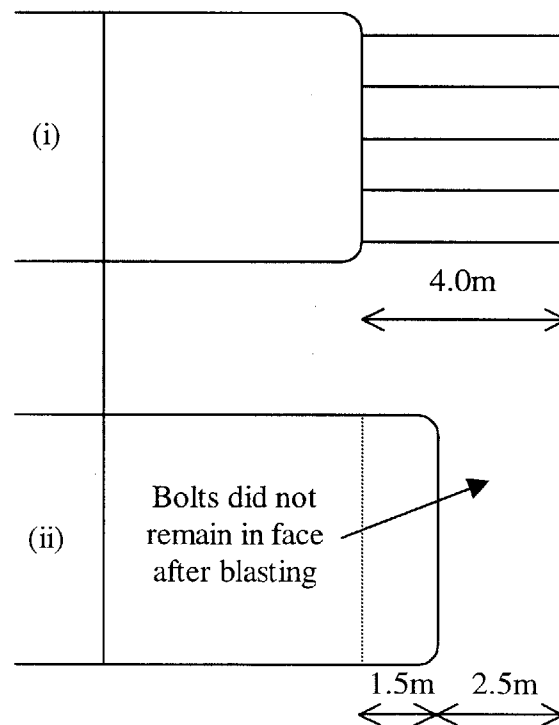


Figure 3.24: Experiences with Fiberglass Bolts and Steel Rebars with Fast Hardening Resin

- The third alternative was bolting the face with Swellex (Manufacturer: Atlas Copco) rock bolts (Figure 3.25, Figure 3.26) proved to be successful.

Swellex is a type of rock bolt that strengthens the rock mass through a combination of friction and mechanical interlock on the interface between the bolt and the rock. The unique feature of Swellex is that it can provide an instant reinforcing action both in hard and soft rocks. The steel bolt is inserted into the drill hole in its folded condition (top of Figure 3.25). Then the head of the bolt is attached to a pump and the bolt is expanded with water pressure (bottom Figure 3.25).

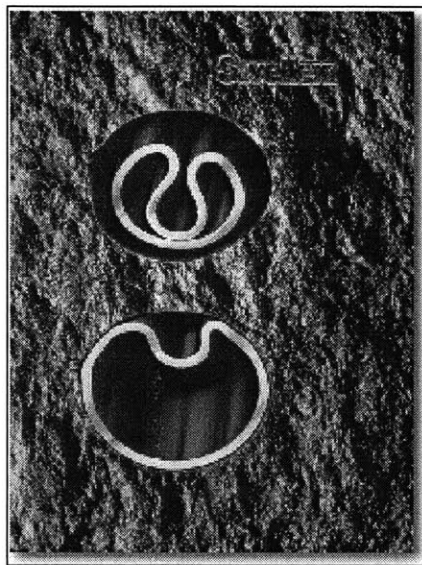


Figure 3.25: Swellex Rock Bolt

Swellex bolts installed in the face behaved ideally, they were cut off with the blast and the remainder stayed in the face. This face stabilization allowed drilling for the next round to continue without additional safety waiting periods. Figure 3.26 shows the bolting pattern as excavation continued in steps of 1.5m rounds. In Figure 3.26 (i) 22 Swellex 4m long bolts are installed. Figure 3.26 (ii) shows the situation after the round was fired. 22 new bolts are installed in Figure 3.26(iii) and the process repeats itself from Figure 3.26 (iv) on.

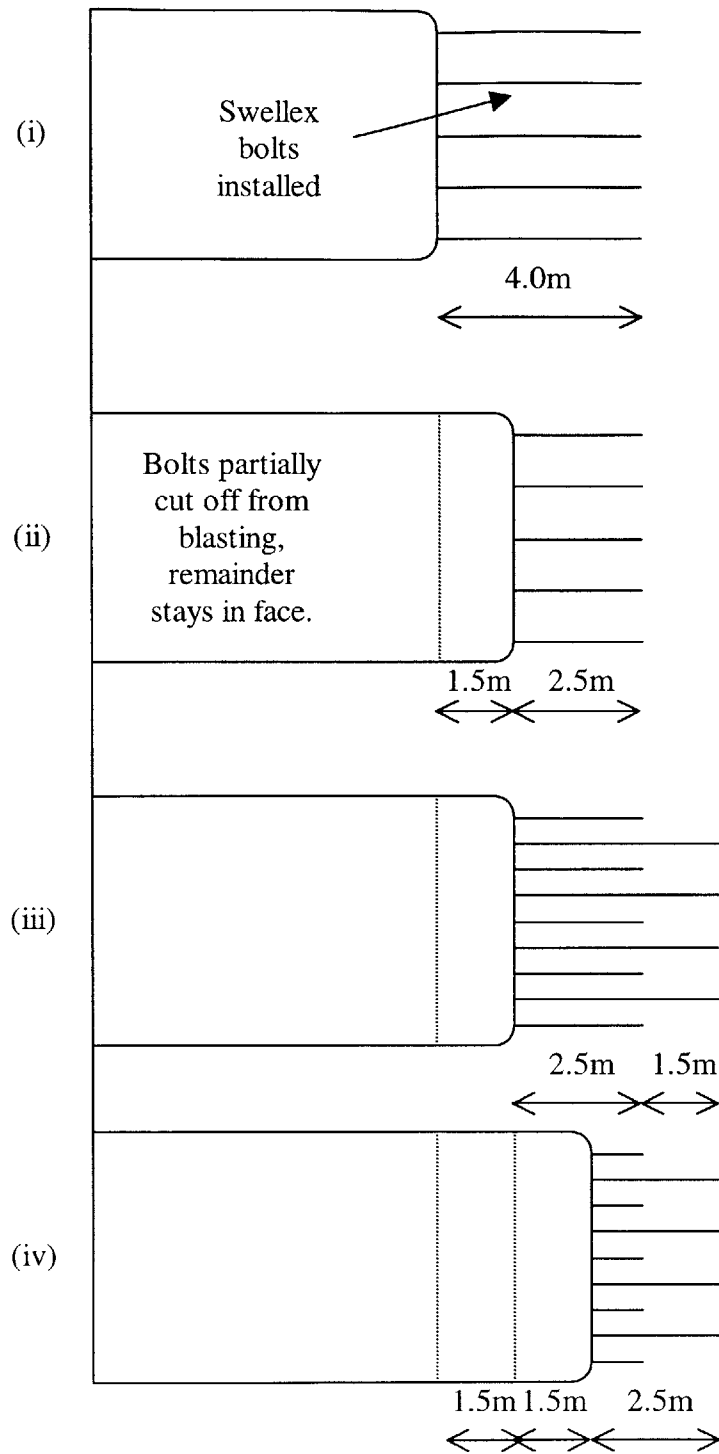


Figure 3.26: Bolting of the Face with Swellex Rock Bolts

Bolting the face with Swellex rock bolts proved to be a significant factor in reducing construction time. It reduced bursting activity and apart from commonly experienced strain bursts due to stress concentrations at the edges of the face (Figure 3.27) rock bursts seized.

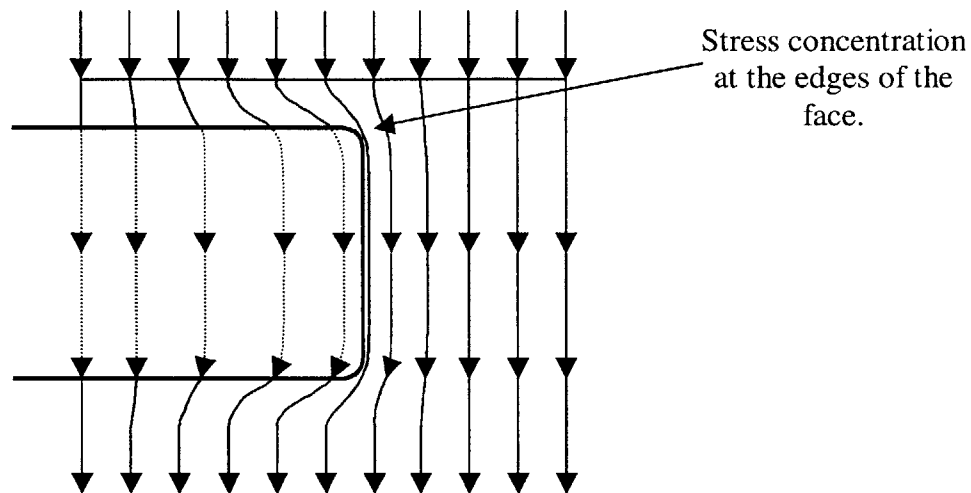


Figure 3.27: Stress Concentrations at the Edges of the Face

From August 1988 more than 4000 4m long Swellex bolts were installed in the face for stabilization. The ease of installation made the contractor switch to Swellex bolts also for the bolting in the roof and about 12500 3m long Swellex bolts were installed for pattern bolting.

Other advantages using Swellex bolts according to the contractor were:

- Ease of installation, especially because the bolt is significantly smaller than the hole at time of installation. → Not subject to problems due to drillhole spalling.
- Immediate support is beneficial, especially for pattern bolting.
- Swellex pumps are easy to install on drilling jumbos and bolting can start immediately after drilling without interruption.

- Bolting is clean without messy cement grout or resin
- Bolt is reliable in water bearing rock

The contractor also noted that installing the Swellex bolts seems to have a similar effect as destress blasting. The rock at the face seemed to be destressed by the large number (22) of boltholes and bolts. The conjectured mechanism is that the rock can first relief stresses due to the new opening (bolthole). Then with the expansion of the Swellex bolts confining stress is generated on the face. Both processes inhibit further failures in the face and therefore made the bolting with Swellex bolts an effective measure against rock bursts.

Yielding bolts are desirable under rock bursting conditions. The yielding bolt would slip and restrain the moving block and when the movement stops it would still have a substantial restraining force. A bolt that fails when its capacity is reached is unsuitable for the high instantaneous loads of rock bursts. Swellex bolts are relatively stiff and can be viewed as non-yielding bolts. However the contractor was able to add a yielding element to the bolt. He placed a wooden block under the face plate to accommodate the strains from fractures in the face. The wooden board also bridges over fractured rock between bolts if one long board is used for two bolts.

After the contractor gained some experience with this bolting method, Bengt Stillborg a Swedish consultant gave the following general design recommendations according to rock burst intensity for the Kan Etsu project:

- **Level 1, Spalling rock, but no serious failures:** Shotcreting of the tunnel face by a remote controlled machine. Bolting of the face with Swellex bolts as required, using the standard washer plate. Length of bolts should be about twice the round length, to pre-stabilize the rock for the next blast (Figure 3.26).
- **Level2, Medium to severe rock bursts:** Application of shotcrete as for level 1. Swellex bolts with wooden board in a systematic pattern. Length of bolts should be about twice the round length, to pre-stabilize the rock for the next blast (Figure 3.26).
- **Level 3, Severe rock bursts:** Analogous to level 2. In addition the shotcrete should be reinforced by wire mesh or use of steel fibers (Dramix®) reinforced concrete..

After establishing and applying the design recommendations the contractor was able to continue construction without any major disturbances. Excavation was completed in October 1989. The tunnel opened for traffic in 1991 and the capacity of the expressway was increased significantly since traffic now flows through two two-lane one-way tunnels.

3.1.3 Ertan Dam, China

Ertan Dam is located in the Sichuan Province of China. It marks the beginning of a multi-project development plan along the Yalong River. China will build 21 power stations along the Yalong River, a tributary of the Yangtze River. This will turn the river into China's largest hydroelectric production base in the 21st Century. Ertan Dam will not only generate much needed electric power, but also relieve the 1000 km downstream Yangtze River from flooding. The 3.4 billion USD Ertan Dam has not received much attention in the western world, since the Three Gorges Dam was in the spotlight, but it is the second-largest power station being built in China. Nevertheless it is an enormous project, which made the relocation of 35,000 people necessary.

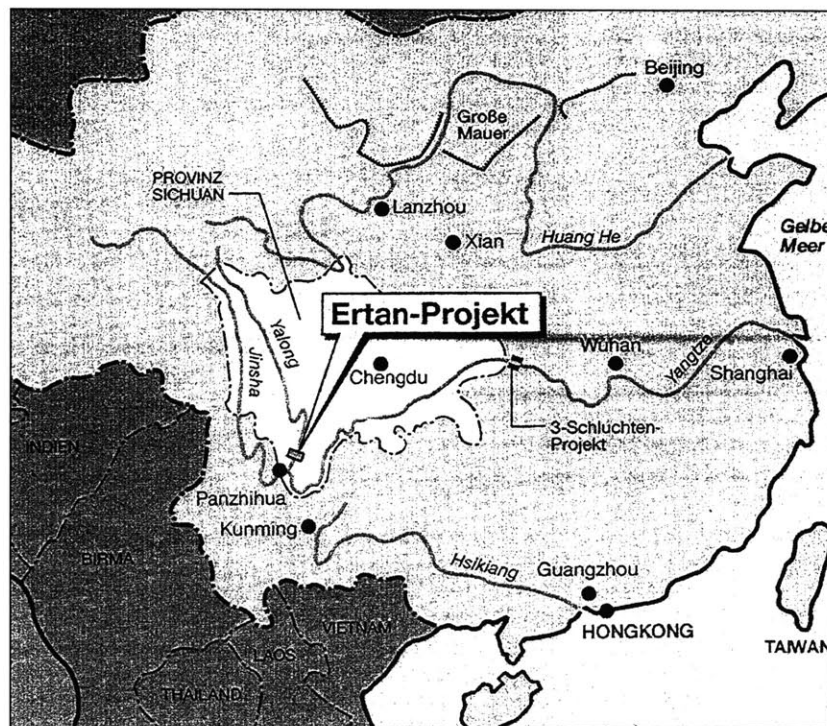


Figure 3.28: Ertan Dam Project, China

Construction of the double arched concrete dam project started in 1991 and was expected to be finished in 2000. It is 240 m high and the underground power facilities have a capacity of 3300 MW and will produce approximately 17000 GWh a year. The project included extensive underground works, such as:

- Two diversion tunnels, 1170 m and 1070 m long and 450 m² in cross section were completed in 1993. These are the world's longest diversion tunnels (Figure 3.29, Figure 3.32).
- Six penstock tunnels and shafts with a total length of 3600 m and 88 m² cross-sections (Figure 3.29, Figure 3.30, Figure 3.32)
- A powerhouse cavern with l x h x w = 280m x 65m x 25.5m (Figure 3.29 to Figure 3.32). This is the largest underground powerhouse ever built in Asia.
- A transformer chamber with l x h x w = 215m x 25m x 18.3m (Figure 3.29 to Figure 3.32).
- A surge chamber with l x h x w = 203m x 65m x 19.8m (Figure 3.29 to Figure 3.32).
- The total volume excavated was 4.13 million m³

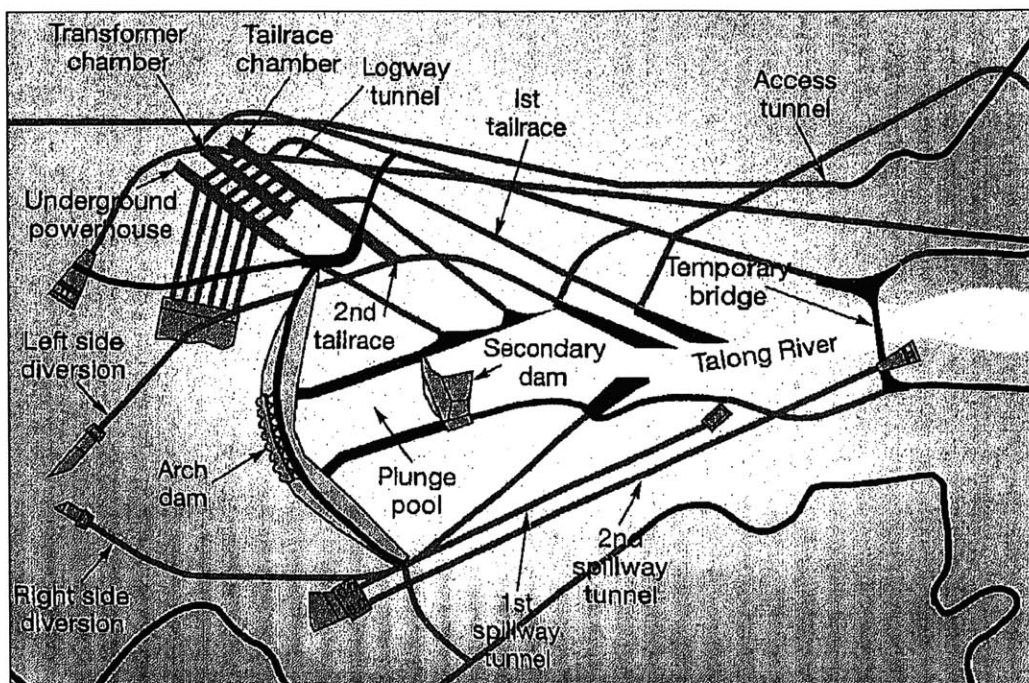


Figure 3.29: Planview of the Ertam Dam Project

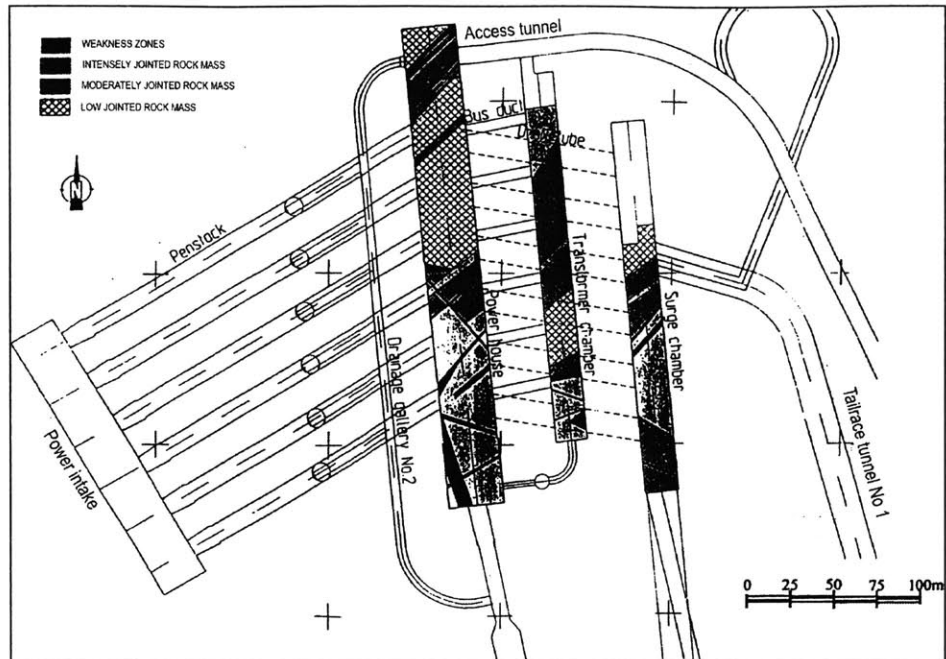


Figure 3.30: Planview of the Ertan Powerhouse Complex

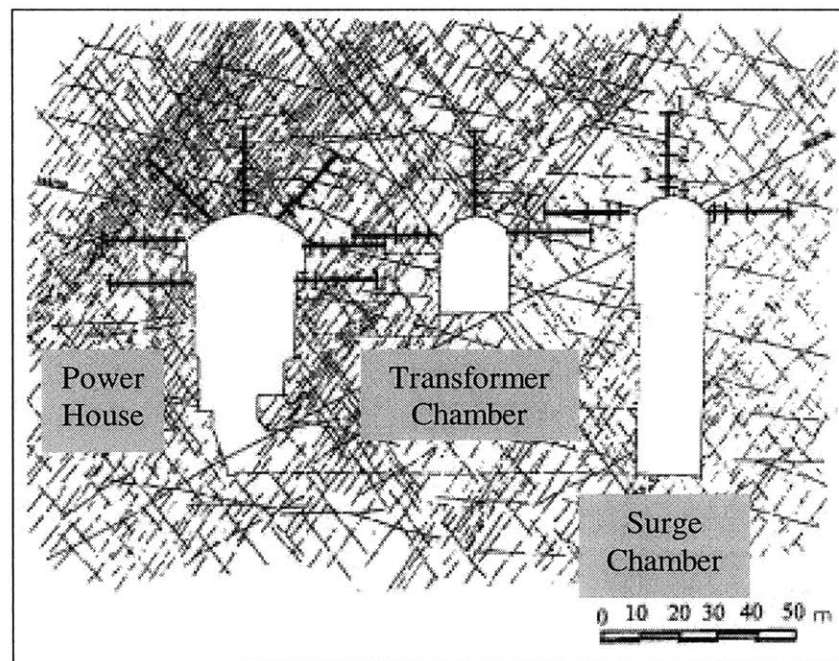


Figure 3.31: Cross Section of Ertan Powerhouse Complex with Instrumentation

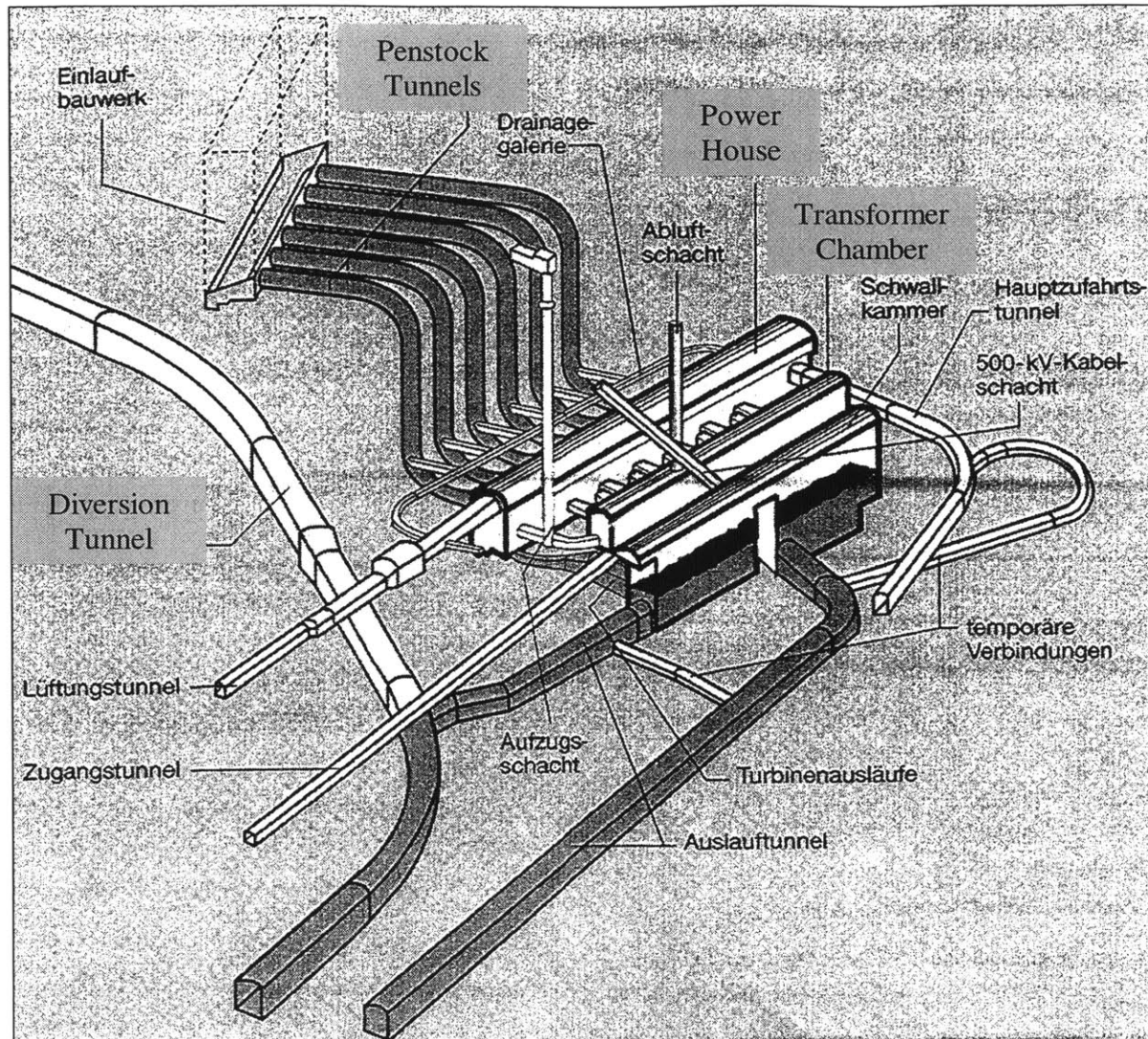


Figure 3.32: 3D view of the Ertan Powerhouse Complex, Diversion and Penstock Tunnel

Drill and blast was chosen for underground excavation. Most excavation work was carried out by a joint venture of Phillip Holzmann and the Chang Jiang Gexhouba Engineering Bureau of China. Excavation was largely finished by 1996.

At the dam site the rock mass is generally composed of igneous crystalline rocks, mainly Syenit, and Gabbro with xenoliths of Basalt. The geological condition is ideally suited for an arch dam. One drawback is the surface quality of the rock mass, which weathers easily in the subtropical climate. This means that extensive short-term and long-term safety measures are necessary. Secondly, high rock stresses in the high quality igneous rock caused significant rock bursts in the underground excavations. The second point is particularly interesting, since the owner instructed the contractor to install lighter support where good rock conditions were encountered. The references do not describe in detail how the support was changed. It is assumed that fewer bolts were installed and shotcrete thickness was reduced. However, rock bursts occurred especially in areas where rock conditions were good. Nevertheless, even with a denser rock bolt pattern and thicker shotcrete as was originally planned the rock bursts would have occurred and created the same problems.

The large scale of the project and the risks and uncertainties involved led to the installation of a complex instrumentation system. To be able to follow the “observational approach” promoted by Terzaghi (1961) with modern methods a “monitoring records management system” (MRMS) was implemented in the project. However it was mainly geared towards movements of the powerhouse complex and the 88 up to 20 m long four-point bored extensometers (Figure 3.31) were of little help to predict the rock burst problem. Serious rock bursts occurred during construction (Figure 3.33, Figure 3.34). Sometimes the contractor was confronted with burst as soon as 12 hours after excavation. In the worst cases these bursts led to an explosive expulsion of rock including the attached shotcrete,

endangering personnel and equipment. The first rock bursts were encountered during construction of the diversion tunnels. In good rock, which prevailed in the diversion tunnels only bolting and local application of standard shotcrete was planned as temporary support. However the stress concentrations of the disturbed stress field around the excavation (Figure 3.21) caused progressive deterioration of the excavated rock surface. The first rock bursts took place at random times: They occurred rarely immediately after blasting before shotcrete was applied. Some occurred after blasting and after shotcrete was in place and some occurred months after temporary support had been installed. The latter makes it obvious, that neither the locally applied shotcrete nor the installed rock bolts were helpful in preventing rock bursts. The additional support work, which was necessary to stabilize the affected areas, had a great impact on the schedule by causing significant delays. It became evident that the temporary support measures (rock bolts and shotcrete layer) had to be replaced by a new support system. The new support system was required predominantly in the large caverns. The roof of the caverns was approximately 250 m below the surface. Due to their immense dimensions, the stress distribution relocation was expected to be greatest here. As a result of their height expelled rocks can fly furthest and endanger large areas in a tunnel at higher energy. Finally more workers in a large workspace and more equipment of higher value would be endangered.



Figure 3.33: Rock Fall due to Rock Burst in Turbine Outlet



Figure 3.34: Rock Burst in Turbine Outlet

Criteria for the characteristics of the new support measures were:

- That the support could be installed as quickly as possible after blasting to provide immediate safety for personnel and equipment on site.
- That, if required, support reinforcement could be carried out easily and in a not too time-consuming manner.
- That it proved sufficient structural stability.

Among the considered solutions was the possibility to install rock bolts with a robot, which operated from a safely supported area. This solution was quickly discarded due to

observations, which showed that even a dense systematic rock bolt pattern could not prevent rock from being expelled during rock bursts.

The selected solution, again, involved a robot. Wet steel fiber reinforced shotcrete was systematically applied immediately after blasting by a robot. As a second measure systematic bolting was carried out. Since the original design of the support only involved conventional wire mesh reinforced shotcrete, the contractor had to set up a test program in order to get approval for the steel fibers. Some steel fibers were immediately discarded because they were severely balling, which would have led to handling difficulties. For all other fibers a mix and spray test was set up.

The shotcrete mix contained:

- 440 kg cement per m³
- Aggregates up to 9.5 mm produced by a crusher from the excavated rock
- River sand to ensure smoother grading and better pumping and spraying characteristics
- Superplasticiser
- Silica dust
- Steel fibers

Fibers, sand and aggregates were added simultaneously at the batching plant. For each fiber product a trial mix of 3 m³ panels was sprayed for compression and bending tests. After 28 days bending strength and the fiber reinforced shotcrete was tested for toughness. 30 mm long Dramix® fibers of 0.5 mm diameter (Figure 3.35) were chosen, because of their simple handling and because only 40 to 45 kg/m³ of fibers satisfied the toughness criteria. Initial estimates required an steel fiber amount of 75 kg/m³.

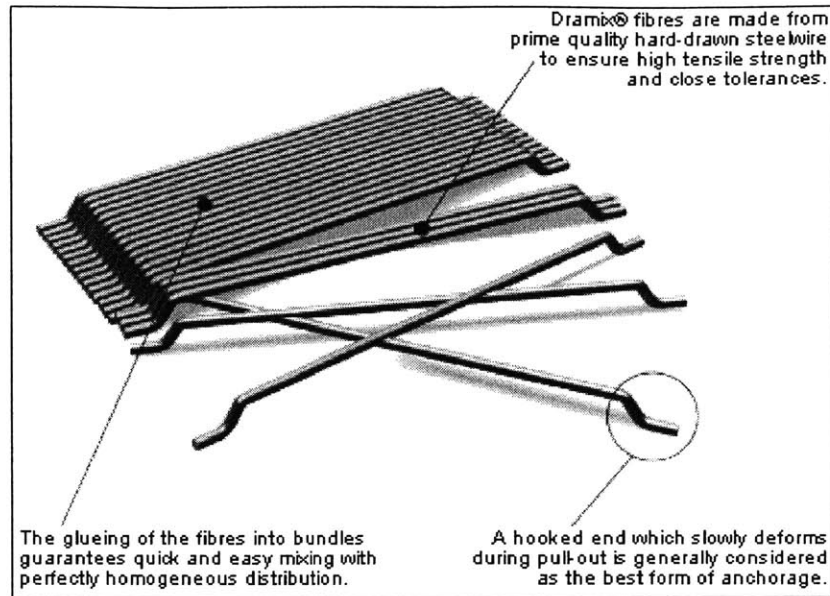


Figure 3.35: Dramix® Steel Fibers

The steel fiber reinforced shotcrete was exclusively applied using the “wet” method. Only in areas where access for the robot was difficult the “dry” method was used and conventional shotcrete was applied by hand spraying. Safer working conditions were the major advantage of steel fiber reinforced shotcrete, which proved to give “sufficient immediate support after excavation providing a safer working environment” (Kuitenbrouwer 1997). The ductility of the fiber-reinforced shotcrete, in essence, provides for warning before failure occurs, and explosive failures are prevented. The fiber-reinforced shotcrete therefore allowed work at the face to continue while additional support was installed in areas where cracks had developed. In addition to the final walls and crown, it was also sprayed on the face as a safety support measure during drilling of the next round. Originally conventional shotcrete was intended for the face, but after the bad experience it was not considered adequate for this function any more.

In the chambers prestressed cable anchors of 15 m to 25 m length and a load capacity of 1750 kN were installed as a third support element after the steel fiber reinforced shotcrete and rock bolting. Figure 3.36 shows the cable anchors in the walls of the powerhouse chamber and the bolting in the roof as originally planned.

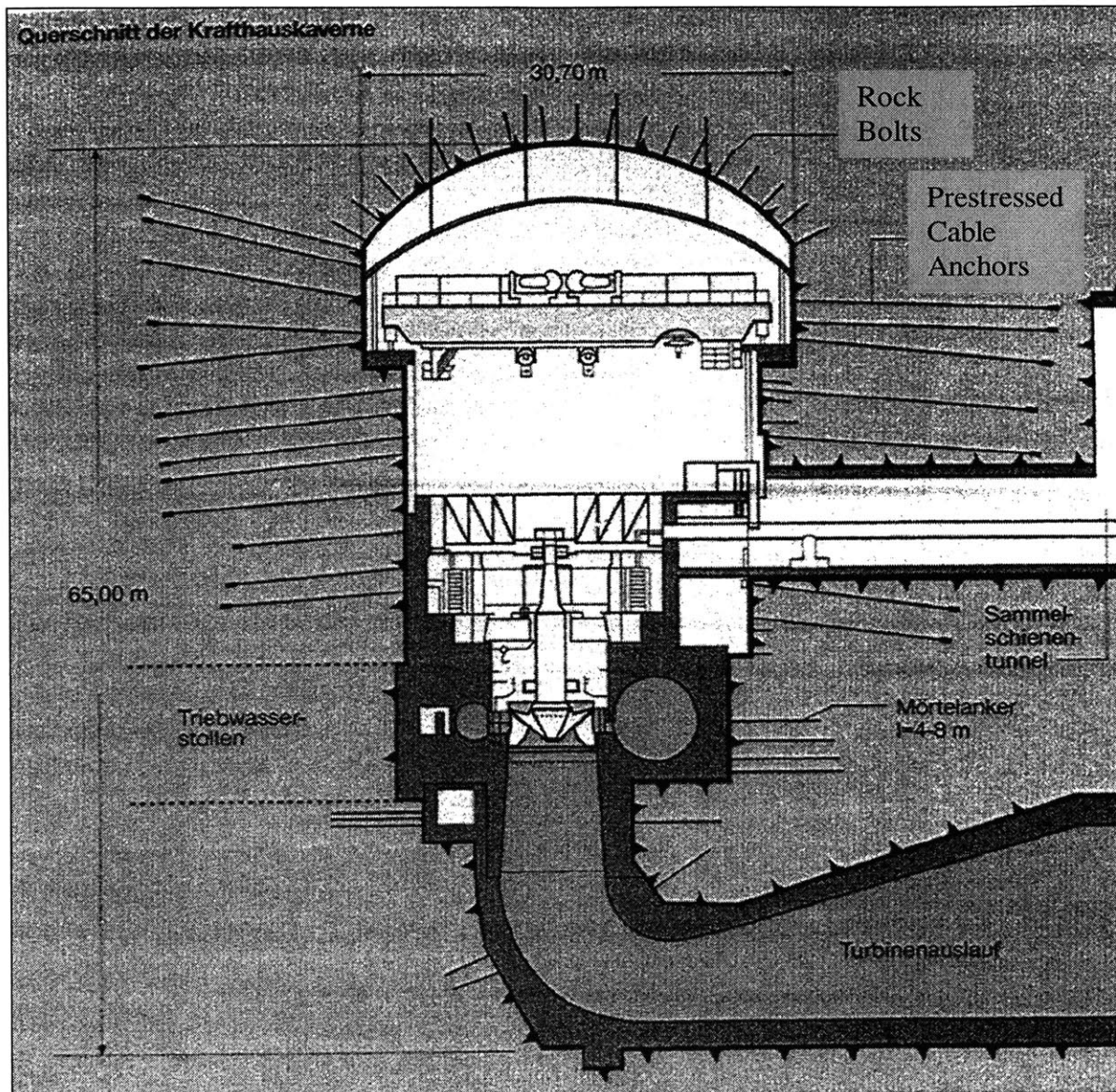


Figure 3.36: Cross Section Powerhouse Cavern with Cable Anchors

Rock bursts and deformations in the caverns caused significant problems, sometimes even failing the cable anchors. A detailed finite element study was carried out and locations of high stresses identified. Figure 3.37 shows the major principal stress σ_1 in N/m^2 . Stress concentrations were found in the upstream upper and downstream lower corners. In these areas additional split set anchors with large base plates and grouted anchors were installed (Figure 3.38). They were needed because they allowed much larger deformations than bolts before failing. In the large caverns a mesh was installed to prevent detached rock from falling. It was affixed permanently by single strand anchors.

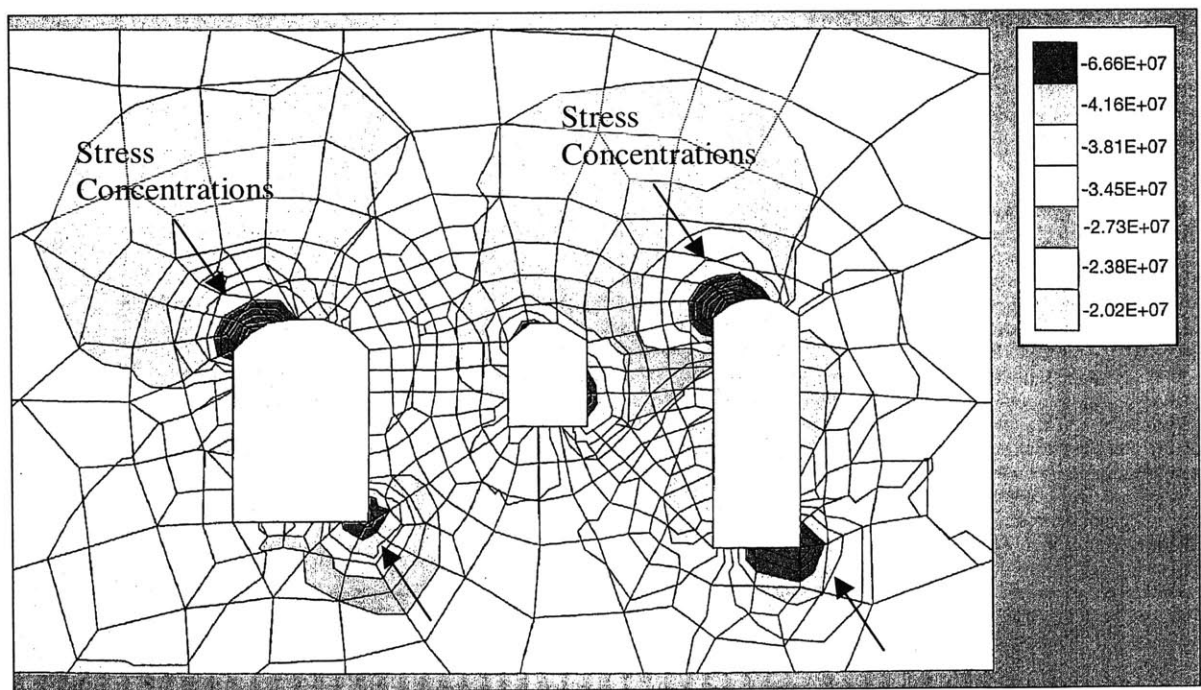


Figure 3.37: Stress Concentrations in the Corners of the Caverns



Figure 3.38: Prestressed Cable Anchors in Caverns

Even with all these support elements rock bursts and high deformation remained a key problem during construction. However, the ductility of the fiber-reinforced shotcrete created a safer work environment reducing the effects and the intensity of abrupt energy releases during the bursts. In addition, the use of fiber-reinforced shotcrete facilitated the implementation of the “observational approach”: Crack development in the lining would indicate that further support measures were needed. The ductile fiber-reinforced shotcrete allowed for enough time to install them before a major failure occurred.

In addition to the rock bursts and high deformation problems several earthquakes occurred during construction. The most severe was recorded at 4.5 on the Richter scale and occurred in September 1995. About 12 hours after the quake, rock bursts occurred in several excavated areas. One can conclude, that even though no measurable movement of the rock mass occurred, the earthquake must have changed the in-situ stress conditions. Rock bursts were only the indirect visible result of stress relief.

Regardless of the significant challenges, the project was not delayed. By eliminating two processes from the construction cycle (spraying of a 50 mm shotcrete layer, installation of the steel mesh, spraying of a second 50 mm or more shotcrete layer to cover the mesh) using fiber-reinforced shotcrete sped up the construction process at the face.

In the case of fiber-reinforced shotcrete only the first process is needed and bolting comes after securing safe working conditions. Time savings were significant. This holds particularly true in areas where partial excavations were performed and only restricted room was available. The higher costs of the steel fiber reinforced shotcrete were offset by avoiding delay in construction. It was possible to make up for a delay of 1.5 years on the critical path and to complete the project ahead of schedule.

3.1.4 New York City

Rock bursts have been reported in several New York City tunnels. The first one was built in 1892 under the East River only 42.4m below ground surface. The rock formations encountered were the Cambro-Ordovician Manhattan Schist in Manhattan, the Cambro-Ordovician Inwood Dolomite under the west channel of the East river and the Precambrian Ravenswold granodiorite in Queens. Charles M Jacobs, the Chief Engineer described the rock burst problems as follows: "...from 46 to 76m east of the west shore of Blackwell's (Roosevelt) Island the rock is of such intensely hard class that it was with the greatest difficulty that the drills could be got to cut it. This section seems to be under some intense local stress that from time to time, even six months since opening, it gives out reports like a pistol shot when the strain cracks the rock, and from time to time masses of it fall without warning. It is only this short length where any such things occur and it is now protected by arching..." The above is typical of descriptions of rock burst in the New York City area. So typical that the term "popping rock" is used instead of "rock burst". One can conclude, that the type of damage mechanism encountered is of the strain bursts type.

The second tunnel where records of rock burst have been found was the City Tunnel No. 2. The 1930 report of the City Aqueduct Department of the New York City Board of Water Supply has a special paragraph titled, "Dangerous Zones in Sound Rock", where rock bursts were described as follows: "Near the bottom of these shafts and in the tunnels 1A N and 2A S popping rock has been encountered in the Yonkers Gneiss. This phenomenon occurs in rock which is apparently perfectly sound and hard (although brittle) but which appears to be under stress from some cause. It has been observed at various elevations

from -39 (Shaft 1A S) to -505 (Shaft 2A S). Thin flakes of rock, of various sizes up to several feet across, loosen or completely detach themselves from rock, which had been proven solid only a short time before. The accompanying noise varies from a report barely audible to one resembling a pistol shot. The popping has occurred so frequently in the tunnel at 2 A S that roof support is necessary to protect the men. Similar popping rock, not pronounced enough to be very dangerous, has been observed in the tunnel at 2A N and also at 15A N tunnel. It has been reported at 10A N tunnel. The last two occurrence are in the Brooklyn Injection Gneiss!" The above description again is typical for strain bursts. The two mentioned rock formations, the Yonkers gneiss and Brooklyn Injection gneiss, are believed to be Precambrian. Both are mainly granitic, almost massive, with little gneissic banding. Some parts of the formations are strictly gneissic and other schistose. Unfortunately the report does not indicate in which parts rock bursts occurred.

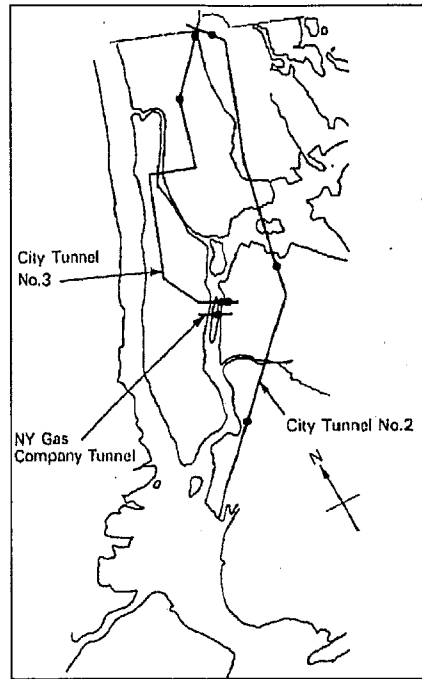


Figure 3.39: Tunnels with Rock Busts in New York

In 1970 work began on the third City water tunnel, which traversed all the metamorphic formations found in the New York City area. At a about 136 m below ground surface, like for the City Tunnel No. 2., rock bursts occurred in the Yonkers gneiss. “Popping sounds” were recorded in the Precambrian Fordham gneiss. Later, in the Manhatten schist no records of “popping sounds” or rock bursts are available. As the tunnel progressed and the rock changed from the Fordham gneiss to Inwood dolomite, the “popping” and rock bursts became common. This has never been encountered before in the dolomite.

Mechanical rock bolts were used to prevent rock burst, without success. The dolomite broke away from around the bearing plate and the rock bolt was left hanging without function. Work was stopped for a few months until the owner and the contractor agreed on measures to be taken to handle the problem. When work started again, a new area of rock bursts was encountered in granite gneiss. Now the popping occurred together with slivers of rock being ejected from the crown. Unlike before where the burst occurred in all parts of the crown, they now came from a distinct line in the crown. The line was a joint parallel to the tunnel. Excavation caused one side of the joint to move, creating a minor moving fault. The contractor supported the area with rib steel. Another problem was that during drilling in the face the rock started to fall down. Up to 20% of the mucking volume for one round had to be removed during drilling. Mechanical rock bolts were replaced by resin grouted rock bolts, which improved working conditions. To protect workers and the integrity of the tunnel both steel rib and wood blocking is used and reported to work as support.

Although New York City has had a 90 year history of strain burst problems, which are due to high in situ stresses, problems have never been encountered in the Manhattan schist, where the deepest tunnels are.

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Chapter 4 Support

4.1 Observations of Support Damage

The damage to support systems and tunnels caused by rock bursts can vary widely. This paragraph shows a series of photos (Kaiser & Tannant, 1999) illustrating damage to support caused by rock bursts. The pictures and descriptions are in order of increasing damage magnitude. All damage shown except for the illustration in Figure 4.6 are due to strain bursting.

Figure 4.1 shows damage from a strain burst event in a tunnel wall triggered by a fault slip event. Dynamic stresses from the remote seismic event caused some fracturing and the wall of the tunnel moved inward exceeding the displacement capacity of the rock bolts.

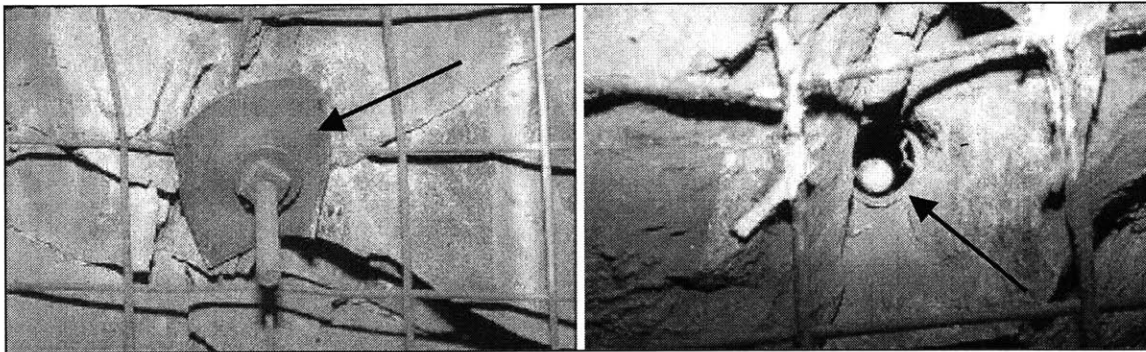


Figure 4.1: Damage to Rock Bolts from Strain Burst (Kaiser & Tannant, 1999)

Both pictures show damage from the same event. The left picture in Figure 4.1 shows a highly loaded mechanically anchored rock bolt. The wire mesh at the right edge of the bolt was cut by the impact of the strain burst. In the right picture one can see a rock bolt that broke near its collar.

Figure 4.2 illustrates strain bursting from overstressed rock around a tunnel. Both pictures show damage from the same event. Small scale spalling can be seen on the left. Higher stresses (right) led to an increased depth of spalling up to 1 m in the roof. From the loosely hanging bolts (right) with broken rock hanging from them one can see that the damaged rock breaks and falls between the bolts. The bolts remain without any function if not used together with shotcrete or wire mesh as a retaining element. In addition the picture (right) shows, that after spalling the roof supports itself.

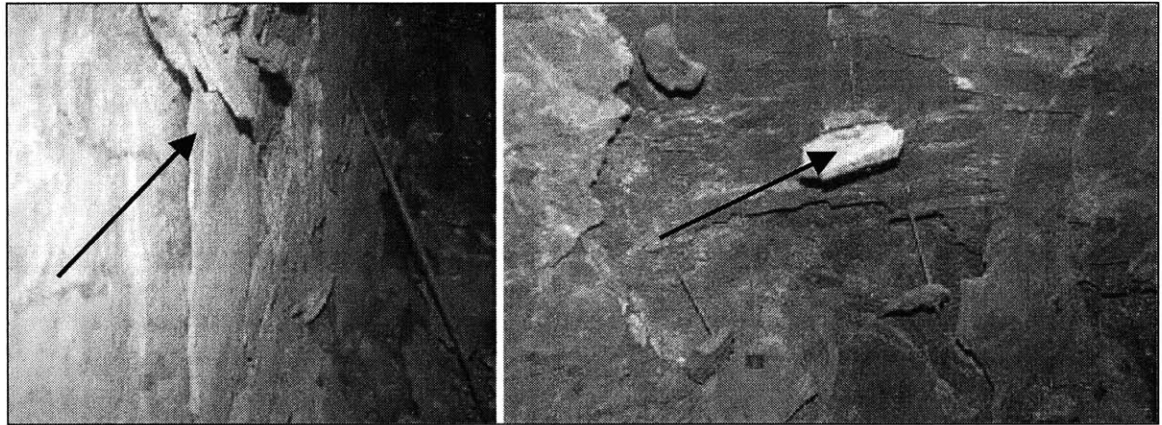


Figure 4.2: Small (left) and Moderate Scale (right) Spalling (Kaiser & Tannant, 1999)

Figure 4.3 depicts moderate levels of damage from two separate strain burst events. On the left one can see that the rock bursting has severely damaged the shotcrete that previously covered the entire pillar. On the right the floor of the tunnel was raised and rock was broken into small blocks.

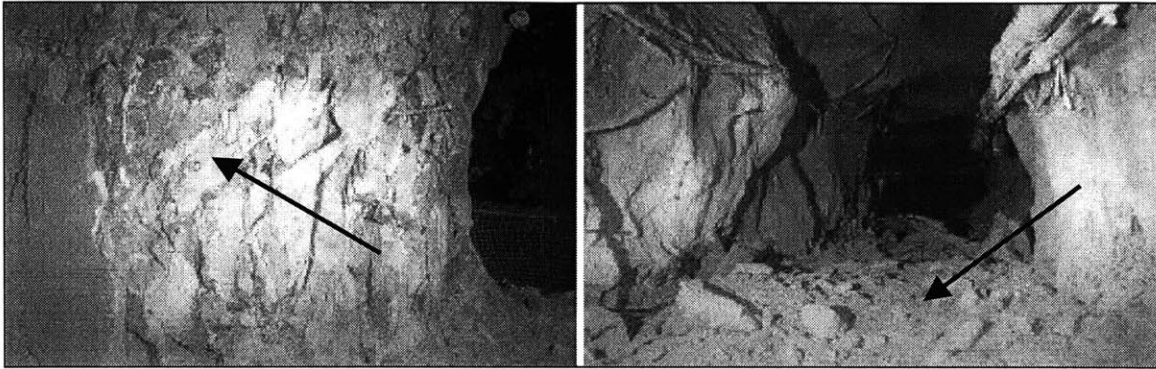


Figure 4.3: Moderate Strain Burst Damage (Kaiser & Tannant, 1999)

In Figure 4.4 a tunnel was completely closed (left) by a heavy strain burst. The support of this heavily supported tunnel consisted of mechanical rock bolts, fully grouted rebar bolts, cable bolts and mesh reinforced shotcrete. The damage was very localized and the tunnel is basically undamaged a few meters from the failure zone. A sheared off grouted rebar in the wall can be seen on the right. This event shows, that it is very difficult to provide sufficient support when severe strain burst damage is involved.

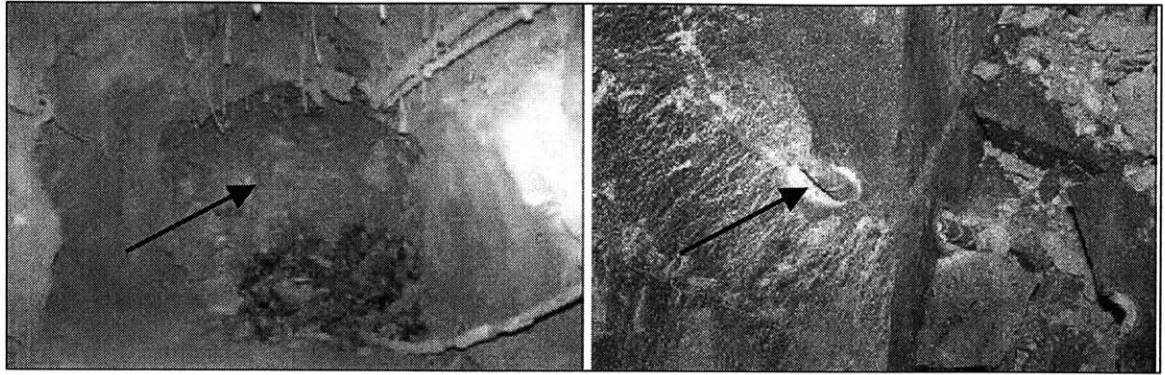


Figure 4.4: Severe Strain Burst Damage (Kaiser & Tannant, 1999)

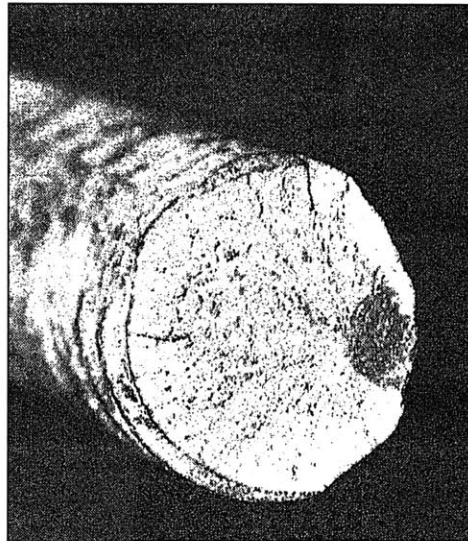


Figure 4.5: Closeup of Fractured Grouted Rebar (Ortlepp, 1992)

Figure 4.6 (left) shows a pile of broken rock from a seismically induced fall of ground. The large sized blocks indicate that is not a strain burst event. Rock blocks from strain bursts tend to be much smaller, because they are highly fractured. This tunnel was supported by cable bolts and rock bolts. Kaiser & Tannant (1999) do not state whether the rock bolts in the illustrations are grouted or mechanical. Depth of failure was 2.5 m to 3 m and exceeded the length of the rock bolts and some cable bolts. Figure 4.6 (right) presents a seismically induced wedge failure. A broken cable bolt can be seen at its right.

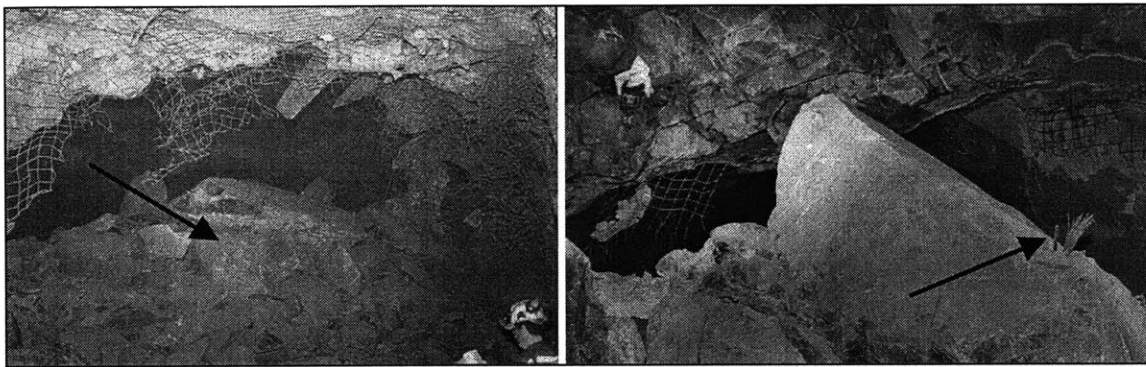


Figure 4.6: Seismically Induced Fall and Wedge Failure (Kaiser & Tannant, 1999)

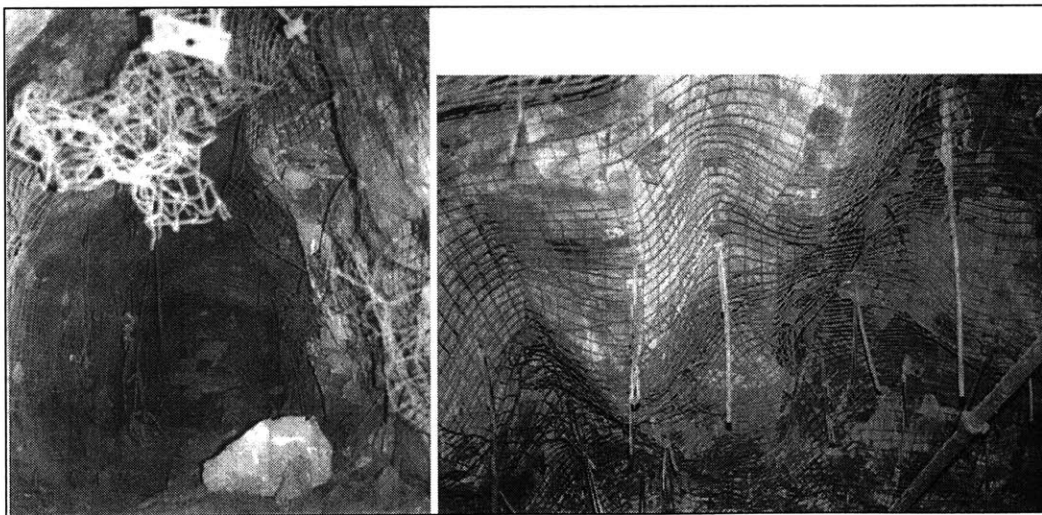


Figure 4.7: (Kaiser & Tannant, 1999)

4.2 Support System Functions

Hoek (1980) defines the function of a support system, as the strengthening of a jointed rock mass by reinforcement to form a rock arch capable of carrying the induced stresses and helping the rock to support itself. Similar thoughts are expressed by McCreath & Kaiser (1992) and are shown in Figure 4.8.

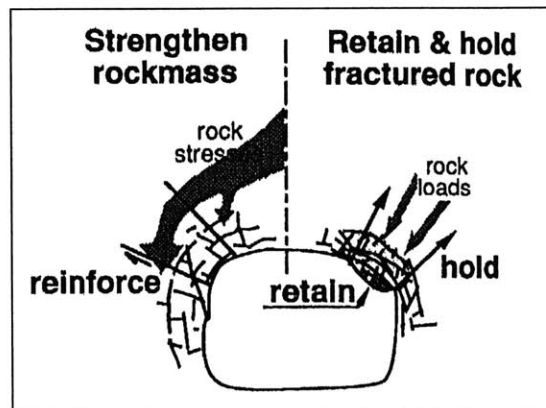


Figure 4.8: Primary Functions of Support (McCreath & Kaiser, 1992)

The reinforcing function can be achieved with fully grouted rebars, which maintain the interlock between rock blocks to minimize loosening. Fully grouted support elements such as cemented dowels or resin grouted deformed rebar fulfil this function efficiently.

In addition it is necessary to retain broken rock and hold it in place by tying it to deeper lying ground. Under normal conditions of relatively low induced stresses the retain & hold function is primarily needed for safety considerations, rather than elementary stability concerns; hence, it can be achieved by the support elements which also serve the reinforcing function. Under very high static or dynamic stresses the retain & hold function plays a more important role in helping to maintain the stability of the cavity. Under very

high stresses significant rock fracturing and substantial deformation occurs. Failure propagates by a process of unraveling the rock mass. Retaining the surface skin of broken rock in place can provide kinematic control over this process. Shah (1992) states that the strength of a jointed or fractured mass increases significantly if a skin of broken rock that is held in place generates confining pressure. In addition, the skin also acts to distribute load, thus helping to protect from impact forces. It has not been verified yet, but might be possible that the fractured rock acts as a damper, dissipating some of the incoming seismic energy.

For high stress conditions this leads to three conclusions regarding support systems exposed to rock burst. First as the stress levels increase, the retain & hold function becomes more critical. Special attention must be paid to the connections between the retaining elements and the holding elements to provide and maintain full area coverage. Secondly as stress increases the reinforce & strengthen function becomes less important, in part because it becomes more difficult to achieve it. The difficulty is to maintain interlock between blocks that fracture at high stress. Third, as imposed deformations increase with increasing stress levels, the support system must be more ductile in order to survive (Wojno et al., 1986).

4.3 Components of Support System

The support system must be designed as a system. One cannot install components isolated from one another. Each component fulfills a designated task and failure of one component most likely causes system failure in case of a rock burst event. What the support system must be capable of and what assignment each component has will be discussed here briefly.

First, the rock mass must be reinforced to minimize further extending the failure. This can be done with grouted rebars. Grouted rebars are fairly stiff and therefore will fail in tension or shear in the rock mass (Figure 4.4, right). It would be wrong to conclude not to use grouted rebars in burst prone ground. Even though they might not serve to hold the rock in place, they are very helpful due to their bulking control function (Kaiser & Tannant, 1999).

Secondly, a ductile support must be added to hold the rock in place. Jager (1992) suggests frictional bolts such as Swellex, or in extreme conditions cone bolts. These bolts have considerable holding capacity over large deformation ranges. They work well together with the rebar bolts, which hold the fractured rock together and have the advantage to enhance the internal frictional energy dissipation (Aglawe, 1999).

To prevent the rock mass from falling down between the bolts as it did happen in Figure 4.2 (right), a third component must be added to the system. An effective retaining component is needed to provide good area coverage. Usually strong mesh reinforced shotcrete or steel strapping is used. Recently steel fiber reinforced shotcrete is used more frequently. It is important that the retaining component can deform together with the yielding bolts. Until the bursting is under control stiff support rings like closed shotcrete

are not recommended (Kaiser & Tannant, 1999). Unreinforced shotcrete is brittle and functions poorly if subjected to dynamic loadings of rock bursts. In severe bursting conditions even adding steel fibers may not improve the toughness of the shotcrete enough to withstand the imposed deformations (Kaiser & Tannant, 1997). For extreme severe cases Ortlepp (1983) suggests to add wire-rope lacing to the support system.

4.3.1 Tendons

A support system for large ground convergence conditions or burst prone ground must contain holding elements (rock bolts, cable, bolts, etc.) that are strong yet capable of sustaining large deformations, i.e., they must absorb as much energy as possible (Kaiser & Tannant, 1995). Most authors emphasize yielding, but one should bear in mind, that high ultimate strength is needed to maintain the capacity of the support after deformation in a new equilibrium. The length of tendons and their spacing pattern are frequently based on broad empirical guidelines. Typical length is in the range of the tunnel diameter, the spacing between tendons is typically 1 m to 2 m and the diameter typically ranges from 12 mm to 20 mm. Where severe stresses are anticipated, pre-stressed rock anchors are used to supplement the tendons.

Stretch limits (Hedley, 1992; Stillborg, 1984) are summarized for six different tendons in Table 4.1. A comparison with the suggested tendons for different levels of support (Figure 4.1) shows that the higher the stretch limit the better the tendon is suited for a high resistance support.

Type of Tendon		Stretch Limit
Rough rebar, standard Swellex, birdcaged cable bolts		< 10 mm
Regular cable bolts		< 25 mm
Mechanical bolts, yielding Swellex		< 50 mm
Debonded cables with a length of:	1 m	< 50 mm
	2 m	< 100 mm
	4 m	< 200 mm
Split set bolts, non lubricated smooth rebar		< 200 mm
Conebolt, lubricated smooth rebar, friction anchored cables		< 500 mm

Table 4.1: Stretch Limits of Tendons (Hedley, 1992 & Stillborg, 1984)

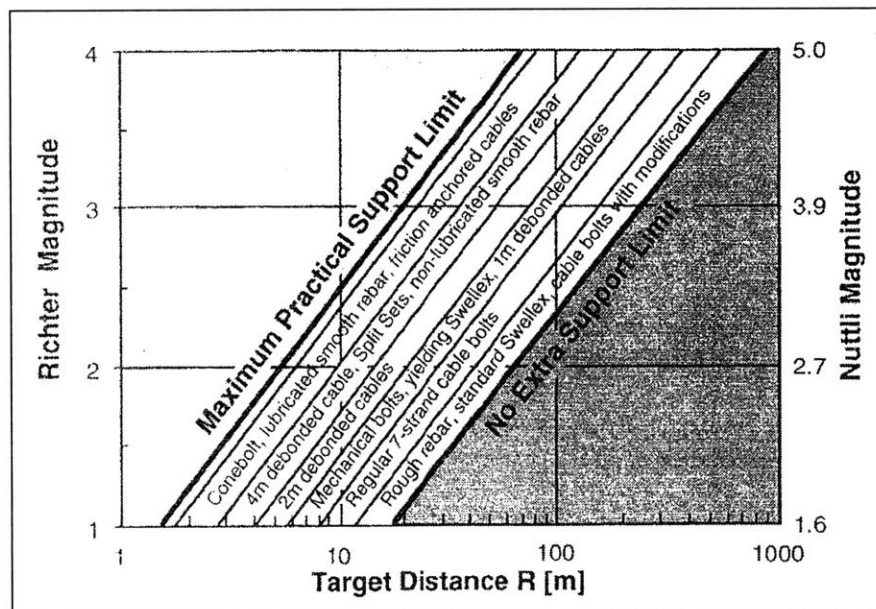


Figure 4.9: Recommended Survival Limits for Holding Elements (Kaiser, 1993)

Ortlepp (1983) has found that the tendon's mode of failure is an important factor in determining its energy dissipation capacity. Ortlepp (1983) confirmed this and the results are shown in Figure 4.10. The two failure modes are the abrupt rupture (i) of the element itself and the breakdown of the bond between the steel and the grout (ii). Figure 4.10 shows that end anchored tendons with no grout (1, 4) have the least energy dissipation capacity. The difference between a grouted and non-grouted bolt can be best seen for bolt 4 and 4a, which fail in failure mode (i) and (ii) respectively. The effect of the bond breakdown is best illustrated in the case of the 22mm rope (7), which has a peak resistance of about 115 kN, and a final frictional resistance that slowly declined from 75 kN to 66 kN. The cement grouted multi rod anchor (5) exhibited an intermediate type of behavior. Rupturing of three out of six rods indicated in Figure 4.10. The remaining three pulled out of the grout with a declining frictional force.

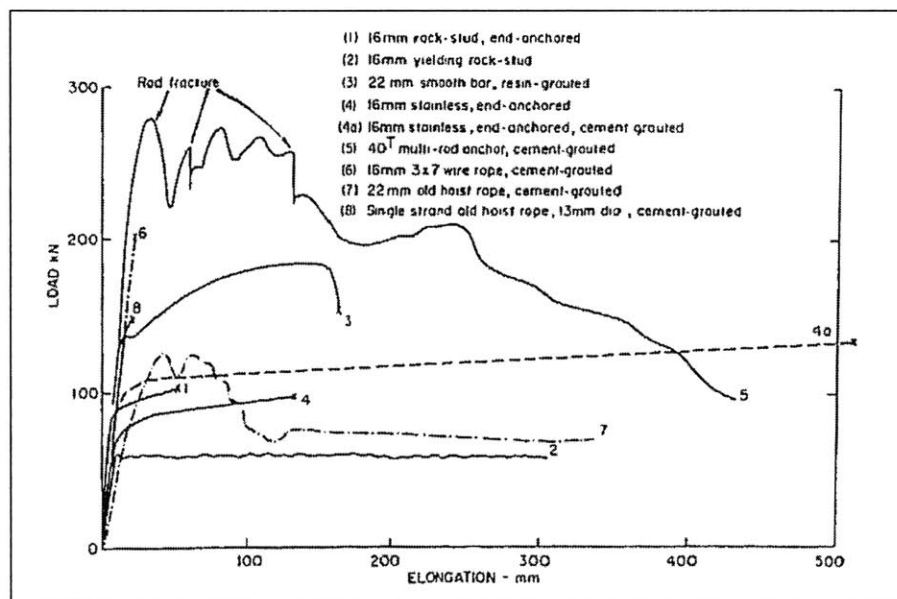


Figure 4.10: Load-Deformation Curves for Various Tendons (Ortlepp, 1983)

From the results Ortlepp (1983) concluded that: “In a dynamic stress situation, if failure of the bond occurred, considerable friction forces would still exist to prevent individual blocks of rock sliding freely along the tendons. Moreover the mesh attached to the tendon ends would prevent a final collapse provided the tendons did not tear or break. On the other hand it is almost certain that a total collapse would result if rupture of the tendons occurred.”

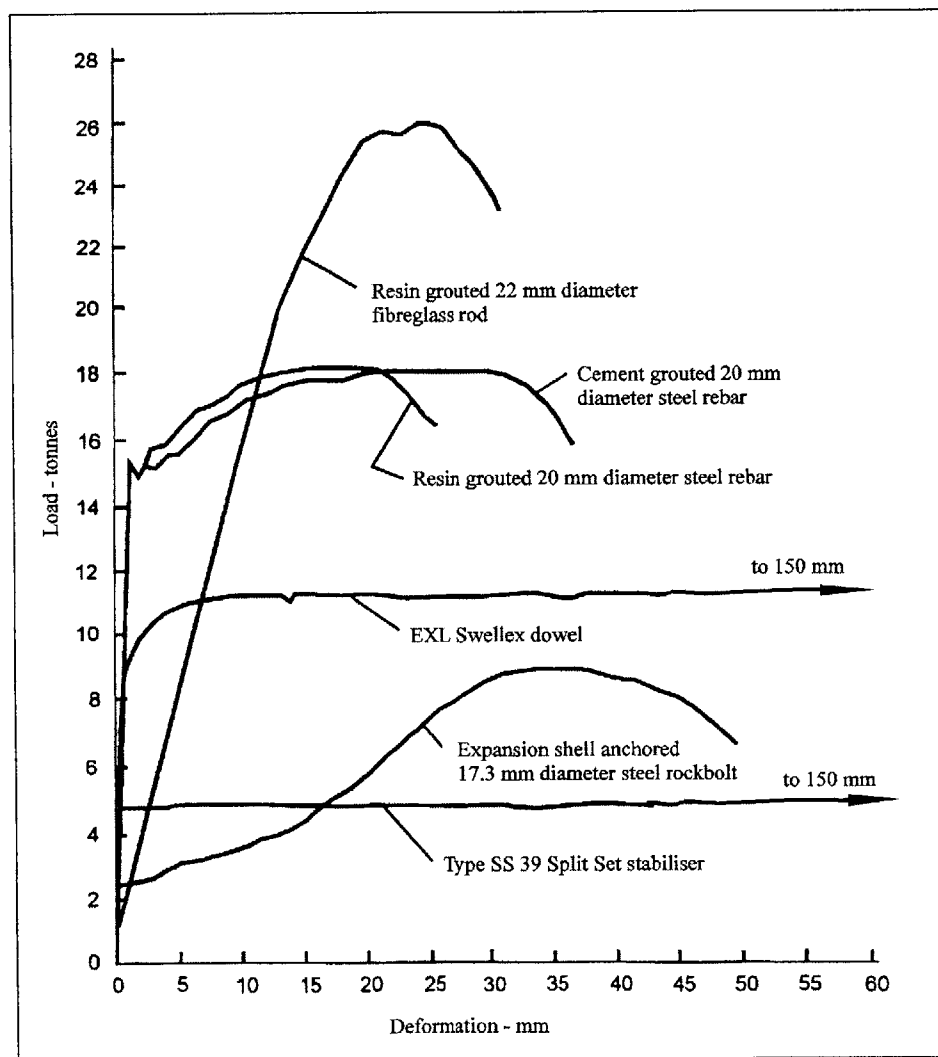


Figure 4.11: Load-Deformation Curves for Various Tendons (Stillborg, 1994)

Stillborg (1994) carried out similar tests, which included Swellex bolts. The results are shown in Figure 4.11 and confirm that bolt types, which tend to get pulled out rather than break abruptly dissipate more energy.

To reduce risk of a failure by abrupt rupture with the total loss of further resistance, it is necessary to limit the bond strength and allow slipping with somewhat reduced frictional resistance. In highly fractured rock the problem of abrupt bolt rupture is not observed as frequently as in fractured rock, since the fracture spacing is small (below the critical bond length) and the bonding forces do not exceed the tendon's strength. Therefore the rock conditions have to be taken into account, when choosing tendon and grout.

4.3.1.1 Mechanically Anchored Rock Bolts

UngROUTED mechanically acting rock bolts have little capacity for energy dissipation. From all bolts discussed here, they are the least suited for rock burst conditions and therefore are not discussed further. Some have a provision for grouting, which improves their behavior during rock bursts as discussed in 4.3.1.

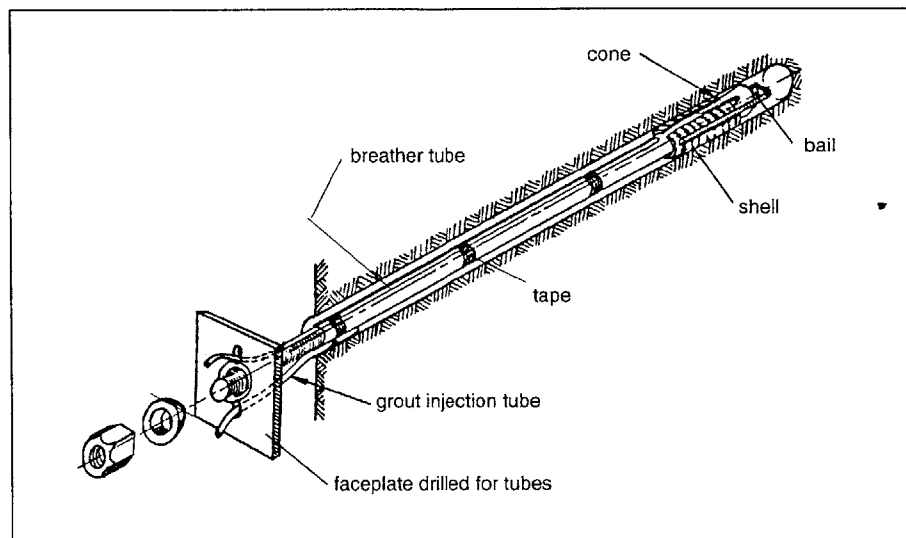


Figure 4.12: Mechanically Anchored Rock Bolt with Provision for Grouting (Hoek, 2000)

4.3.1.2 Cement Grouted Rebars

Being simple and cheap and with an ultimate capacity of about 180 kN cement grouted rebars are very common in tunneling. Lack of ductility makes a support consisting merely of grouted rebar suitable only for relatively few small-scale strain bursts. Only mechanically anchored bolts and resin grouted bolts have a more disadvantageous stress strain behavior. Another disadvantage, like all cement grouted bolts, is that they become effective only after the grout has hardened. As with all steel bolts they cause problems during drilling the next round when applied in the face. During blasting the entire bolt is ejected with the rock mass, leaving the face exposed unprotected. The energy dissipated by a grouted rebar can be determined directly by its length before and after failure. From many observations of rebars failed in rock bursts it is evident that, if the grouting is done “right”, little de-bonding from the rock occurs and the elongation and therefore the energy dissipation of a grouted rebar is negligible. For a ejection rock burst event this brittle support behaves in a manner as if there were no support at all.

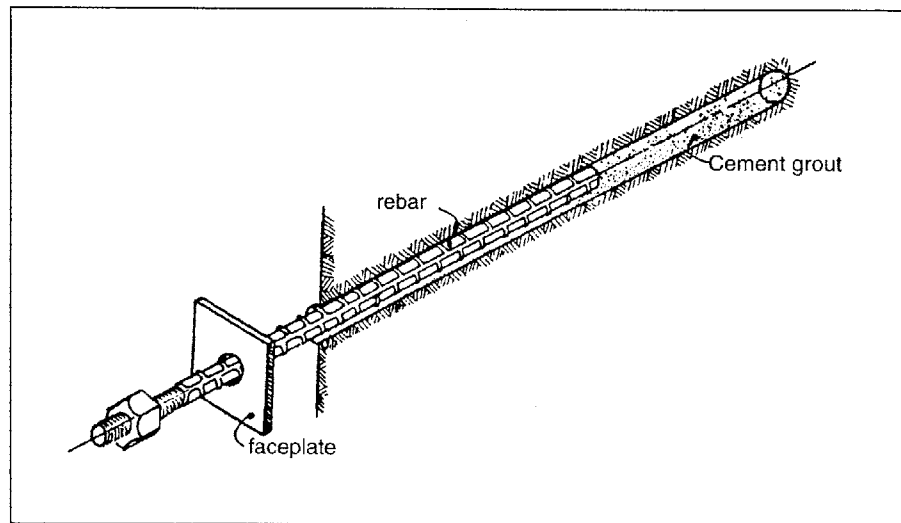


Figure 4.13: Cement Grouted Rebar (Hoek, 2000)

4.3.1.3 Cement Grouted Cables

Compared to cement grouted rebar, cement grouted cable has the advantage of failing gradually, thus being more ductile and dissipating more energy. A typical cable failure can be seen in Figure 4.10 (5 and 7) with sudden drops in load when a strand fails. High strength cable with high yielding forces is suited well for rock burst conditions. A drawback is the dependence on grout quality, which is difficult to control.

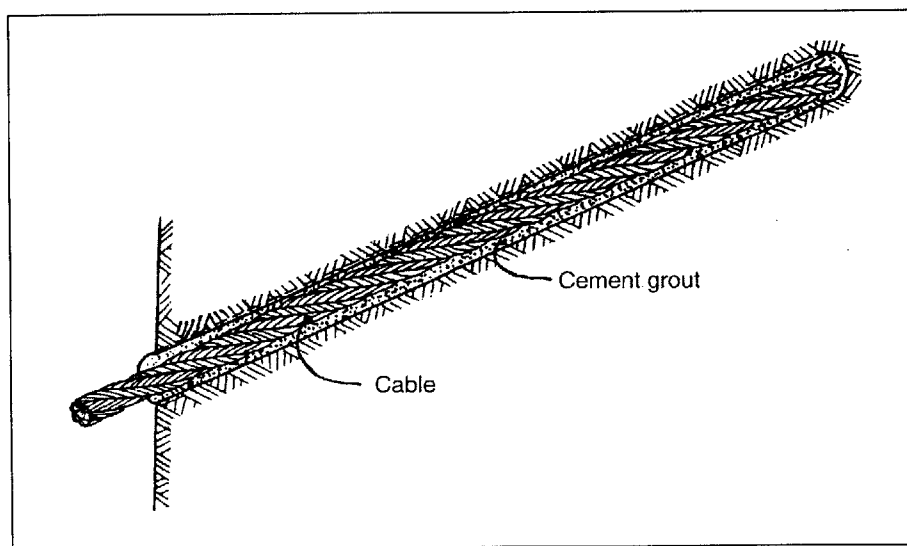


Figure 4.14: Grouted Cable (Hoek, 2000)

4.3.1.4 Resin Grouted Rebars

Resin grouted rebar differs from cement grouted rebars only in the grout used. The advantage of resin-grouted rebars is the faster hardening of the resin. As can be seen from Figure 4.11 their behavior is even more brittle than that of cement grouted rebars, thus being even less suited for rock burst conditions.

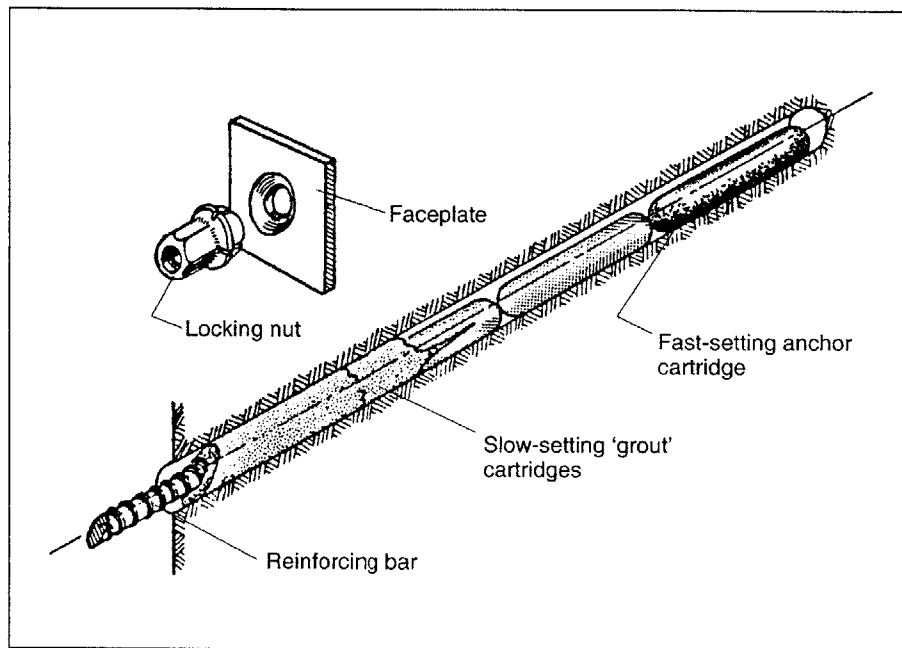


Figure 4.15: Resin Grouted Rebar (Hoek, 2000)

4.3.1.5 Split Sets Bolts

Split set bolts are useful in mild rock burst environments, because they will slip rather than rupture (Figure 4.11). The system consists of a slotted high strength tube and a faceplate. It is installed by pushing it into a slightly undersized hole to generate the radial spring force. The bolt provides frictional anchorage along its entire length, is quick to install and provides immediate support. Corrosion remains a problem of the system and hence it is not frequently used in civil engineering.

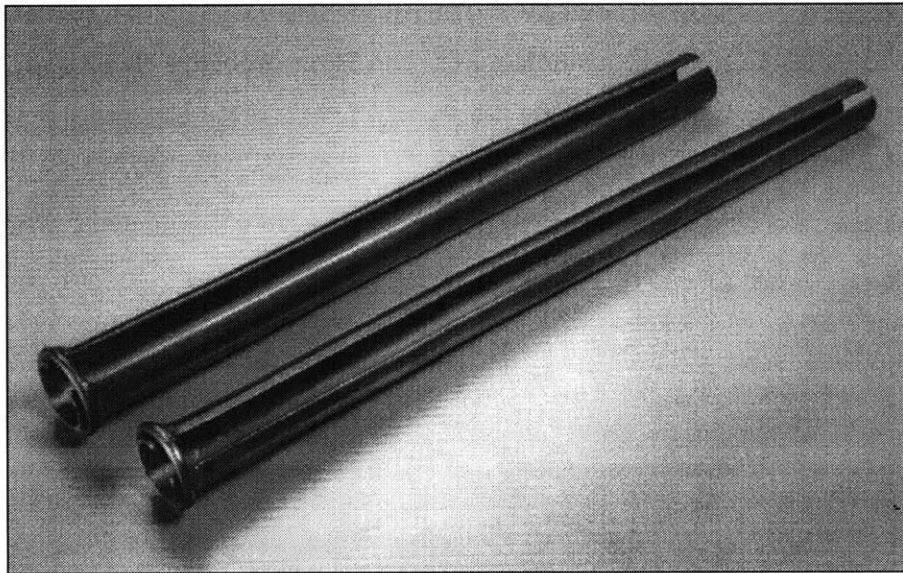


Figure 4.16: Split Set Bolt (Ingersol-Rand)

4.3.1.6 Swellex Bolts

Swellex bolts are mentioned frequently in the context of rock bursts, it should be pointed out though, that several types exist and only the “Yielding Swellex” and “Yielding Super Swellex” are capable of dissipating enough energy. Their advantageous behavior is due to their frictional resistance over the full borehole length. Like split set bolts they do not fail by abrupt rupture, but rather get “pulled out”. Swellex bolts are quickly installed and provide immediate support. Another advantage in rock burst conditions is that they can be easily installed by automatic rock bolters, thus no personnel is exposed to dangerous areas. According to Atlas Copco, the manufacturer, corrosion problems have been solved with coating.

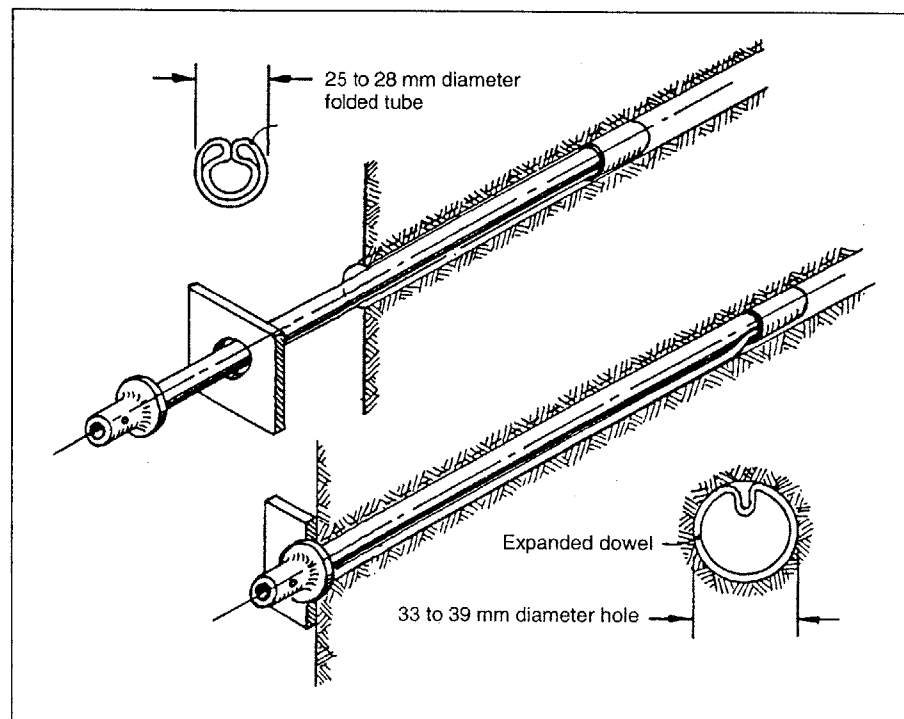


Figure 4.17: Swellex Bolts (Hoek, 2000)

4.3.1.7 Cone Bolts

Cone bolts are a relatively new system developed for the mining industry. They are a viable yielding tendon with a performance that is very well suitable for rock burst conditions. The concept and design is shown in Figure 4.18. The tendon is installed and grouted into the borehole the usual way. However, to allow it to yield by being pulled through the grout, no bonding between the grout and the tendon can be permitted. This is achieved by coating the tendon with wax. The principle of operation of the cone bolt is through displacement of the dilating rock, in which the tendon is installed, to the tendon via the bearing plate. This leads to pulling the conical end piece through the grout. The yield force is generated by a combination of the friction between the grout and cone and the force required to compress the grout.

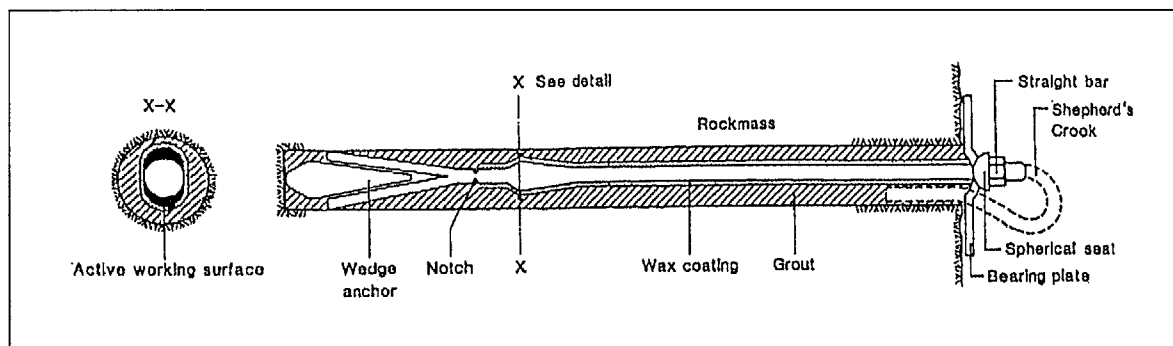


Figure 4.18: Cone Bolt Yielding Tendon (Jager, 1992)

The size and shape of the cone determines the yield force (Figure 4.19), the stiffness and the vulnerability to quality of grouting and fracture openings in the grout. The force generated by the cone is proportional to the difference between the cross sectional area of the cone and the cross sectional area of the bar.

The cone bolt allows for the higher displacements than any of the previously discussed bolts. It does not fail even if the anchor plate has moved as much as 60 cm from it's original position, making it one of the best tendons available for rock burst conditions.

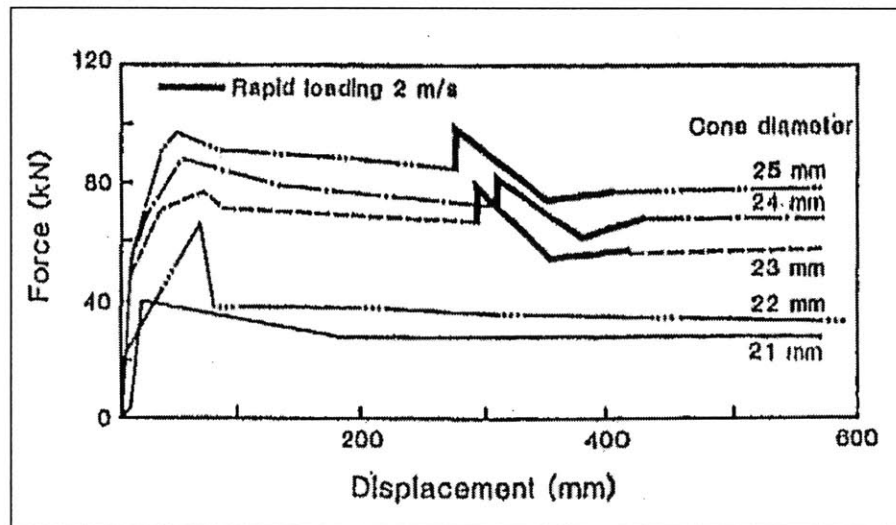


Figure 4.19: Yield Force-Displacement Curves

4.3.1.8 Yielding Hydraulic Props

Yielding and rapid yielding hydraulic props were designed for the mining industry. They are suitable for high-energy rock bursts, since they allow for large displacements at high velocities. Figure 4.20 shows a 3 m/s hydraulic prop with load spreader. The rapid yielding hydraulic prop shown has three force levels. The adjustable setting force (120 kN to 160 kN), the slow yield force (170 kN to 200 kN) and the rapid yield force (400 kN to 500 kN).

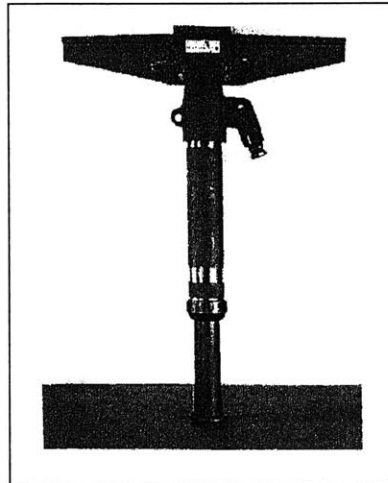


Figure 4.20: Hydraulic Prop with Load Spreader

Figure 4.21 Shows these settings and the relating support function. So far hydraulic props have only been used in mining tunnels. The reasons seem to be the more severe rock burst conditions encountered in mines and the prior use of these bolts in areas, which were exposed to blasting.

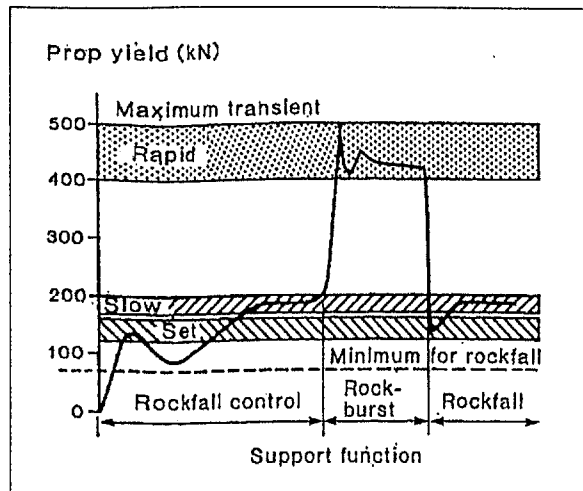


Figure 4.21: Forces Specified for Various Functional Requirements

4.3.2 Shell

4.3.2.1 Mesh

Mesh is rarely used in civil engineering tunneling as permanent support. For mining tunnels mesh often provides a very efficient and ductile support. Especially woven mesh, which does not have any limitations in regard to aperture size or strength of wire, can be an option when a ductile high strength support is required in extreme conditions. Figure 4.22 shows the characteristics of different types of mesh. Given that these curves are valid for specific wire strengths they do not allow one to infer absolute strength, they give an indication of mesh's ductility, however.

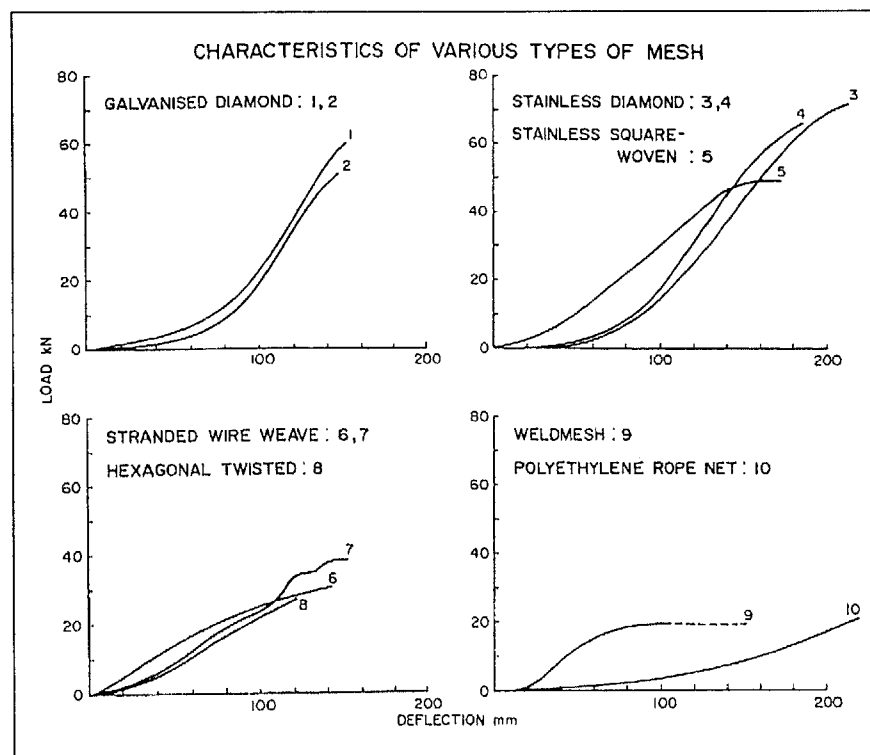


Figure 4.22: Characteristics of Mesh (Ortlepp, 1983)

4.3.2.2 Shotcrete

The use of shotcrete, which is a relatively rigid support, appears to contradict the point of view that a soft or yielding support should be used in burst prone areas. However the desirable characteristic of shotcrete is that it distributes locally applied loads over a much larger area and serves as a retaining component. For low energy rock bursts with localized displacements shotcrete can provide sufficient support due to its distributive ability (Langville & Burtney, 1992). In high-energy bursting conditions the stiffness of the support system is critical and shotcreted areas might not perform any better than unshotcreted areas (Morrison, 1990).

The mode of behavior of shotcrete and its effect on stabilizing the ground when applied in relatively thin layers (<10 cm) to irregular rock surfaces is not entirely understood even under static conditions (Kaiser, 1993). Claiming full understanding of the behavior under dynamic loading would be overstepping the present knowledge base by far. Several authors proposed explanations for shotcrete behavior. The following reflects their current interpretation of how shotcrete might interact with the rock mass and when or if it is of assistance as a support component to control rock burst damage. Since the function of shotcrete differs with the rock it is applied to details will be explained later.

- It acts as a support ring providing internal support pressure. Convergence-confinement concept; Hoek, 1980
- It provides punch resistance against blocks of rock.. Shear strength of shotcrete; Fernandez (1976) & Holmgren (1983)

- It acts as a “supermesh” (see 4.3.2.2.3). McCreath & Kaiser, 1992
- It controls the rock mass dilatation by tangential and/or lateral shear resistance.

A wide range of ground conditions can be encountered in practice. The two extremes are heavily fractured and unfractured rock, which will be discussed separately below.

4.3.2.2.1 Shotcrete on Unfractured Rock

Intact rock fails by fracturing near the cavity. Strain bursts are the prevailing damage mechanism. The rock is broken into small blocks and ejected with high ejection velocities. It literally self explodes. It is a sudden failure with no pre-announcement and pre-event dilatation before the peak strength of the rock is exceeded. The shotcrete skin will deform and will be ejected with the rock unless it acts as a retaining element.

As a result shotcrete does not considerably change the stability of the cavity. Shotcrete applied to intact rock is not an effective support element unless stable rock mass fracturing can take place behind the shotcrete. However, if a ductile shotcrete membrane, held by yielding tendons, is properly integrated into a support system it can act as a retaining element (Kaiser, 1993).

4.3.2.2.2 Shotcrete on Fractured Rock

If a fractured rock mass is exposed to dynamic or static loads the fracturing process continues and the rock mass dilates into the cavity. The rock blocks do not only move radially into the opening, but also tangentially relative to each other (Figure 4.1). The shotcrete layer resists the shear movement at the shotcrete rock interface. It stops the rock blocks from moving freely, even though the radial pressure at the interface is small.

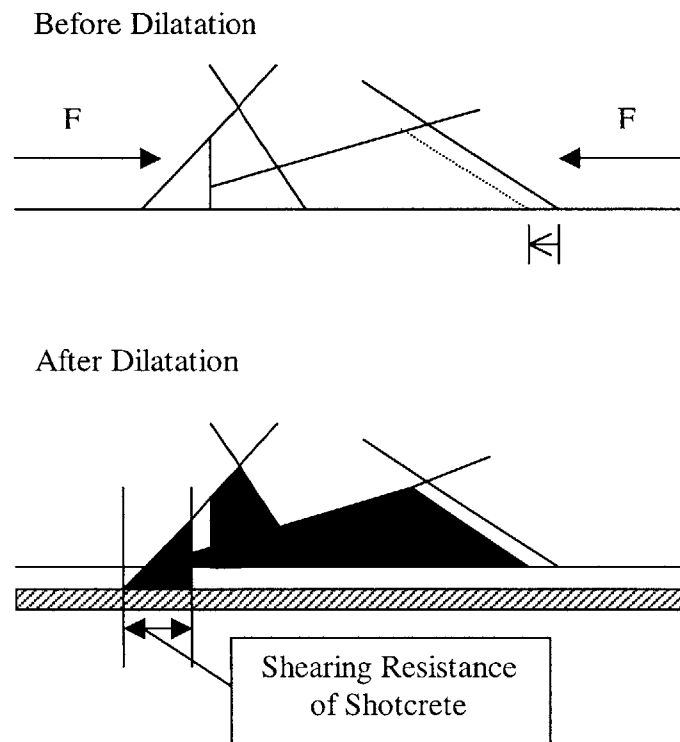


Figure 4.23: Relative Movement of Rock and Shotcrete

To understand how shotcrete can be used to effectively retain rock burst damage one has to understand the relative movements during dilation and that the shearing resistance of shotcrete is a key factor in its retaining ability. The shotcrete's tensile and compressive strength determine its shearing resistance, since the induced shear is resisted by tensile and compressive forces. Hence for large deformations its tensile strength is an important characteristic.

In addition to the shear strength of the shotcrete itself the shear resistance at the concrete-rock interface plays an important role in strengthening the rock mass. An analogy with a soil triaxial test or uniaxial compression test explains how. In these tests friction between the specimen and the loading platens increases the apparent strength of the specimen. Increasing the height to width ratio of the specimen, the zone influenced by shear on the specimen-load platen interface becomes relatively smaller and the specimen fails at lower loads (Brown & Gonano, 1974). Hudson et al. (1972) found that the strength increase for a specimen with shear interfaces is much more intense at large strains (0.3%). For a shape factor $d/h = 4$ (d/h = diameter/height) the strength was 12 times higher than without shear at the interface. Therefore shotcrete that causes shear resistance at the interface can significantly strengthen the rock mass up to the depth of the shear influence zone.

The consequence that shotcrete strengthens fractured rock is not really striking. Much less intuitive, but more important for rock burst containment is that it makes the rock mass near the surface more ductile. Kaiser (1993) used data from Hudson et al. (1972) and Brown & Gonano (1974) to relate post peak stiffness λ (Figure 4.24) to the percentage of “surface with shear” (Figure 4.25). Post peak stiffness λ is the slope $\Delta\sigma / \Delta\epsilon$ of the post peak strength curve from an unconfined compression test, which represents the unloading stiffness of the rock. The unloading stiffness of the testing machine is defined as k . To record post peak stiffness λ of a rock specimen one needs a stiff loading machine. This means that

$|k| > |\lambda|$ is a condition for stability. One can apply an analogy of the testing machine to rock bursts; burst prone rock represents the specimen and the surrounding rock mass the loading machine.

Thus if $|k| > |\lambda|$ no strain bursts will occur. From Figure 4.25 one can see that with increasing shear on the interface $|\lambda|$ becomes smaller, thus the rock becomes more ductile reducing its proneness to burst. Up to date no field study has been carried out yet to prove these laboratory observations. Experience in mining generally supports the results.

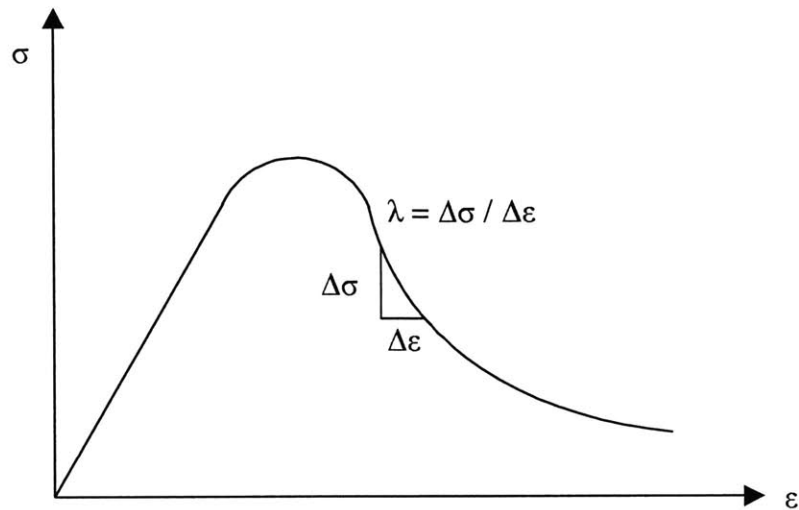


Figure 4.24: Post peak stiffness

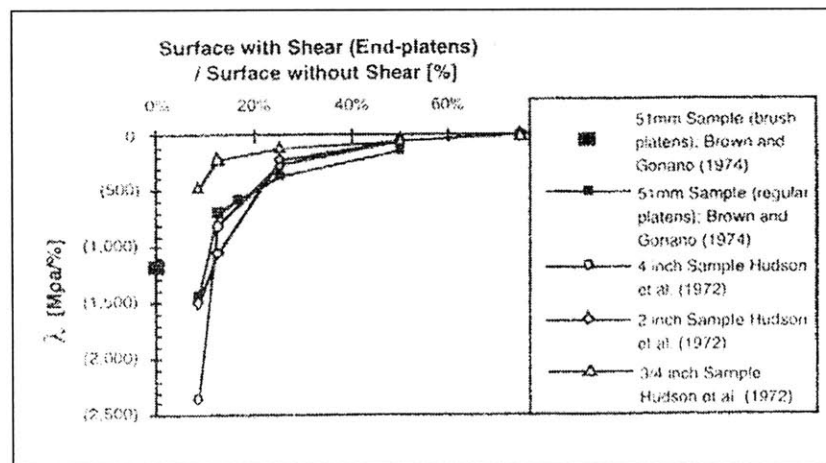


Figure 4.25: Effect of Interface Shear on Post Peak Stiffness (Kaiser, 1993)

4.3.2.2.3 Shotcrete as a Supermesh

McCreath & Kaiser (1992) identified the integrity of a support system to retain small and large pieces of broken rock as one of the key functions of a support system designed to contain rock burst damage. They concluded that if reinforced shotcrete was held in place by suitable tendons it would act as “supermesh” and provided excellent retaining capability.

Shotcrete has several positive characteristics that make it a “supermesh”. It is a closed skin and retains even small pieces of rock, which prevents injuries and initiation of an undesired loosening process. It protects the mesh from corrosion and local impact so breakage of individual wires, which would allow individual blocks to fall out, is prevented. Shotcrete also strengthens the mesh at overlaps. It distributes impact loads among more holding elements due to its bending stiffness. Since shotcrete reduces lateral movements the load applied on the bolts is almost pure tension, the type of loading they are designed for. If shotcrete is used and the mesh is well connected (Figure 4.26) to the rock bolts, the full load can be transferred to the bolt and the frequently observed mesh tearing over the plate is prevented.

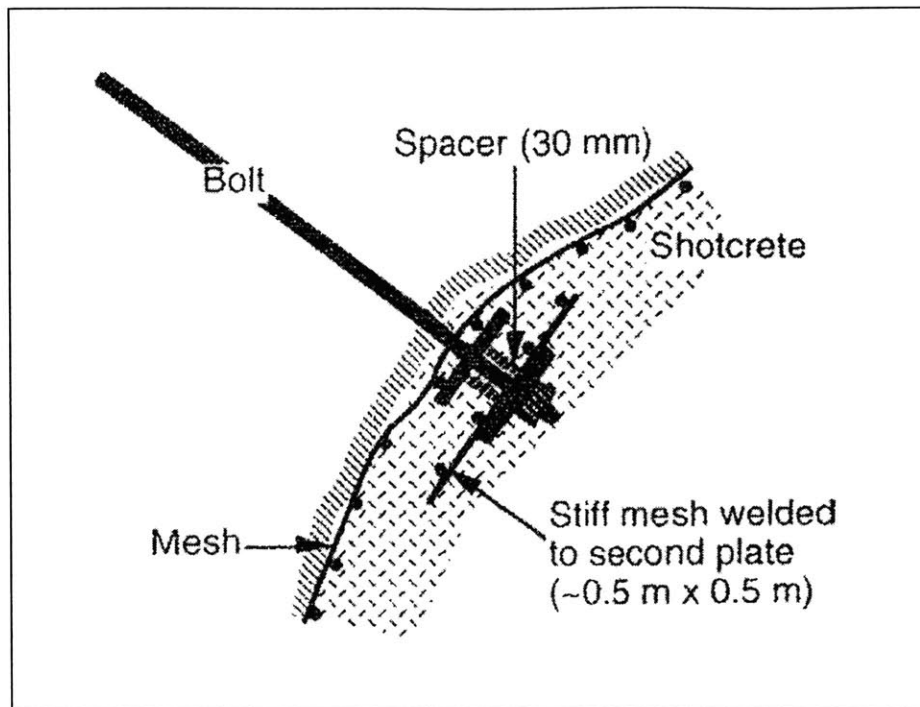


Figure 4.26: Detail Mesh-Rock Bolt Connection (Kaiser, 1993)

A “supermesh” with the described characteristics is a very good retaining system. If it stays in contact with the rock mass it also provides the shear and interlock interface necessary in fractured ground (Kaiser 1993).

4.4 Support Systems Used in Mines

The following will give a brief overview of support systems used under rock burst conditions mainly in South African and Canadian mines because of their substantial experience in dealing with these conditions. Support systems used in tunnels will be excluded since they were described with the case studies in Chapter 3. The purpose of the overview is to illustrate the available options and explain local practice. They are not design suggestions for similar conditions, because support practices reflect various factors such as particular ground conditions, excavation approach, available materials and specific experience of the personnel. Nevertheless, preliminary guidelines for design can be found in Chapter 4.6 and Figure 4.37. The systems used are divided into “support levels” ranging from level 1 to level 5 as support increases.

4.4.1 Canadian Practice

Support systems in Canadian mines are selected according to rock mass quality and static stress conditions. The approach and thinking is generally based on the reinforce & strengthen model of a support system’s function. If rock bursts are expected it is common practice to increase the strength of the support system with additional support elements. Under extreme conditions cable lacing might be ultimately installed, but cost may prevent this from being implemented. Cable lacing (Figure 4.27) is wire rope, that is attached to the tendons. Frequently it is laced through the looped ends of the shepherd’s crooks (Figure 4.28). Even though the lacing method is somewhat determined by the density and pattern of the tendons, a diamond pattern as in Figure 4.27 is usually used.

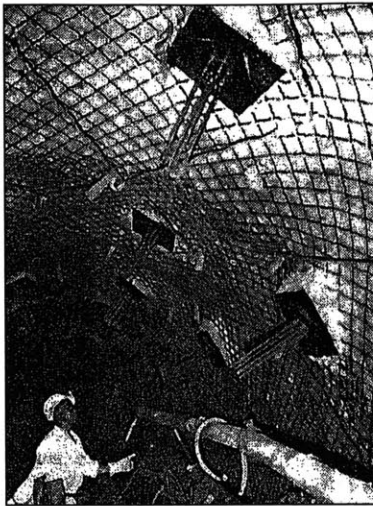


Figure 4.27: Lacing

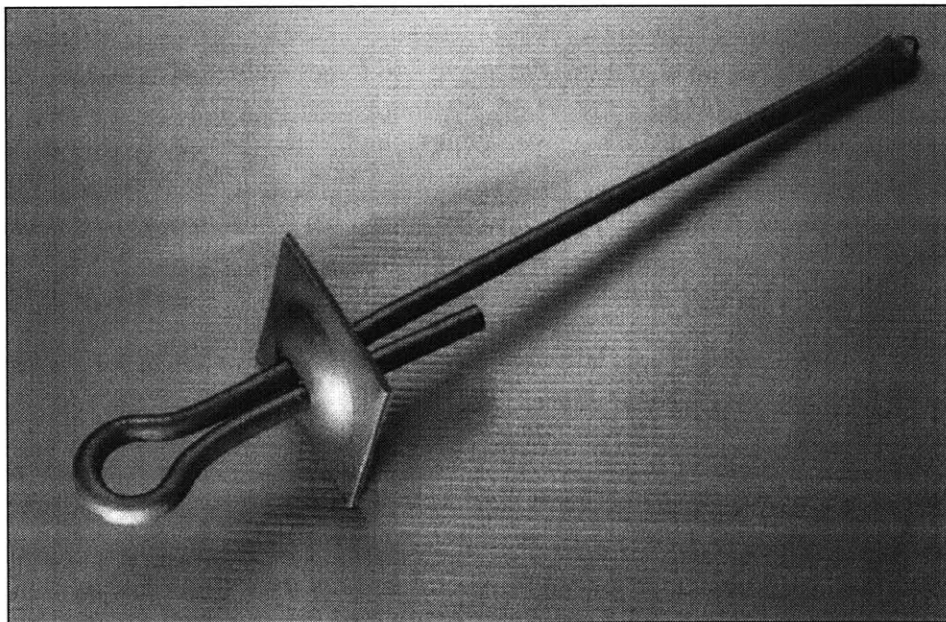


Figure 4.28: Shepherd's Crook

In a simplified way, Canadian support practice can be considered to consist of three levels:

Level 1: Mechanical pattern bolting with end anchored ungrouted bolts. Usually welded wire screen or chain-link mesh. Typically 1.8 m long, 16 mm diameter mechanical bolts in a 1.2 m by 0.75 m pattern. The typical welded wire screen is #6 gauge with 10 cm by 10 cm openings, sometimes link mesh with 5 cm by 5 cm is used.

Level 2: Mechanical pattern bolting like in level 1. In addition 20 mm diameter resin grouted deformed reinforcing 1.8 m to 2.4 m bars, placed between the mechanical bolts. If rock bursting risk is perceived the mesh or screen is often carried down the walls to about 1 m above the ground. Split set bolts might be placed in the walls. With increasing rock burst risk the bolt/rebar pattern is more closely spaced and heavier mesh and longer rebars are used.

Level 3: On rare occasions cable lacing similar to the one used in South Africa (see below) was placed in high-risk areas. Typically 7 strand 16 mm diameter twisted cable is used in a 1.5 m by 1.2 m pattern. There is a large cost increase from level 2 to level 3, especially when placing the support while rehabilitating an area after a burst.

In the last 10 years more and more shotcrete was used at all levels, replacing the welded wire screen and chain link mesh.

4.4.2 South African Practice

Brummer (1991) reports that support in South African mines usually is installed in 2 phases. Primary support is installed during the drill and muck cycle and secondary support some time after the face has advanced about 50 m by a specialist crew. The support used reflects that most mine openings in South Africa are in fractured rock with very high static stresses (Wojno et al., 1986). The support elements must accommodate significant deformations under these static conditions. Elements which are suitable for these static conditions also have characteristics desirable under bursting conditions, such as their ductility and the ability to retain broken rock. The approach of simply adding more of the same support as soon as bursting conditions are encountered is better suited to the South African system than to the Canadian systems, because the latter are much more likely to have a higher degree of burst resistance due to their ability to deform better. The appropriate support level in South Africa is typically chosen according to the stress in the rock and the change of stress over the lifetime of the cavity. Few tunnels are primarily designed to resist rock bursts. If more intensive bursting conditions are encountered the level of support is simply increased. In a simplified way, one can describe three typical levels of support in South African mines:

Level 1: 2 m by 2 m diamond pattern of 16 mm diameter and 2.4 m long shepherd's crooks (smooth or deformed rebar with eye formed by bent over leg) as primary support.

Level 2: In addition an intermediate pattern of shepherd's crooks with a final spacing of 1 m by 1 m. Chain link mesh with 3 mm wires and 75 mm openings. Lacing cable to protect mesh from being torn off its connections to the shepherd's crooks.

Level 3: As above with increased density of shepherd's crooks. Grouted cables up to 6 m long might be added. For permanent openings or very poor rock shotcrete and welded wire screen might be added. Practitioners consider shotcrete too brittle for inward deformations of up to 50 cm under static conditions.

4.4.3 U.S.A., Chilean and Australian Practice

In the U.S. (Coeur d'Alene, Idaho) the primary support pattern used consists of 0.9 m pattern of 2.4 m long high strength grouted resin deformed bars (Dywidag). Chain link mesh with split set bolts is installed in an intermediate pattern. This support, which consists of stiff reinforcing elements (Dywidag bars) and ductile retaining/holding elements (chain link mesh and split set bolts), has survived well under bursting conditions.

In Chile (El Teniente mine) a 0.75 m pattern of cement grouted high strength deformed bars of variable length with chain-link mesh (10 cm openings) is used in burst prone areas. Shotcreting is standard practice in all burst prone sections of the mine.

Limited experience is available in Australia and support is rather conventional similar to the Canadian. Under conditions of large static deformations cables with a debonded length at the collar are used to provide ductility.

4.4.4 Summary of Behavior of Support Elements under Bursting Conditions in Mines

If the full aerial integrity is not maintained immediate and violent unraveling of the rock leads to the collapse of the opening. Badly designed connections between the retaining elements and the holding elements have led to a chain reaction sequence of failure. Mechanical end anchored bolts lose their effectiveness as a reinforcing element due to anchor slippage and loss of bearing under the plate. Grouted deformed rebar often fails immediately under the plate in bursting conditions, but it has been observed that the rebar is continuing to play a useful strengthening/reinforcing role. Chain link mesh behaves in a more ductile manner than welded wire screen mesh, but is more difficult to install than the stiff welded wire screen mesh from below the screened area. Chain link mesh that is not held by large plates or cable lacing will be torn off the holding elements. Shotcrete as a “supermesh” has high initial stiffness and but might have limited ultimate ductility. Split set bolts have uncertain holding capacity and because of corrosion are not used for permanent support. Their ductility in the axial direction however is quite good and installation in broken ground is easy. Swellex bolts are generally considered to be too stiff to provide ductility under bursting conditions. However, some successful trials have been made in Japan (Stillborg & Hamrin, 1990) and Canada.

4.5 Desired Support Characteristics for Bursting Ground

Based on the South African experience it is possible to design support systems that can survive fairly large rock burst events. However these cable-lacing support systems are labor intensive to install and therefore extremely expensive, they also slow down the tunneling progress. A desirable support therefore lies between the cable lacing and the support system used for not burst prone ground. The following ideal characteristics are a guideline for designing support systems placed in bursting ground:

1. High initial stiffness of reinforcing elements for strengthening the rock mass.
2. As far as possible, continuation of the reinforcement function under conditions of large imposed deformations.
3. Improvement of the support system ductility. This is desired particularly in the retaining/holding function. Ductility can be added by using highly deformable mesh or by yielding tendons or both.
4. Preservation of full area coverage of the retaining and holding elements and their connection.
5. Efficient integration of the elements of lower level support systems into higher level support systems. It is advantageous to have a support system for rock burst conditions that can use the same elements as the “regular” design.
6. Presence of multiple lines of defense within the range of elements used for the support system.

7. Wide range of applicability for all levels of support systems, to avoid the need for accurate prediction of rock burst potential and/or detailed determination of the rock mass characteristics.
8. Practicability and acceptance by the workers.

4.6 Design of Support Systems for Rock Burst Environments

4.6.1 Current Design Rationales for Support in Burst Prone Ground

St. John & Zahrah (1987) carried out a comprehensive study on the seismic design of subsurface excavations and underground structures. They reviewed how structures are designed against earthquake hazards. Primarily the design consists of estimating a design magnitude based on an empirical site-specific relationship defining the intensity of the ground movement as a function of the source distance. They use peak particle velocity (ppv) and peak particle acceleration (ppa) to describe the intensity of ground motion. They recognized that damage can be caused by three different factors: fault slip, ground failure and shaking. Their suggestion is that any seismic design should start with simple empirical methods, since most of the data needed for a detailed analysis of damage is often not available. If justified, a more detailed analysis can be carried out later. Owen & Scholl (1981) established “no damage limits” for $ppa < 0.2 \text{ g}$ and $ppv < 200 \text{ mm/s}$ and “major damage limits” for $ppa > 0.5 \text{ g}$ and $ppv > 900 \text{ mm/s}$ from an empirical database. McGarr (1983) states that ppv correlates better with damage than ppa. In summary the design logic of St. John & Zahrah (1987) consists of four steps:

1. The seismic activity center is defined by location and character.
2. An empirical relationship to asses the energy decrease drom source to target is established
3. Damage mechanisms and limiting values for damage related parameters are determined.
4. The structure is designed to resist the dynamic loads.

Ortlepp (1992) too developed a design procedure that consists of four steps:

1. Determination of the most likely mode(s) of failure.
2. Establishment of a representative model for these failure mode(s) to determine how failure can be prevented.
3. Determination of material strength for the model(s)
4. Selection of a safety factor, often empirically determined, to ensure an acceptably low probability of failure.

As practiced in geotechnical engineering, the identification of probable failure modes as the basis for a rational engineering approach to support design for the containment of rock bursts is extremely important. It requires that the design methodologies must be grouped according to the expected damage mechanisms and limits of applicability must be well defined to ensure that the most appropriate model is chosen for design (Kaiser, 1993). From a support design perspective Ortlepp (1992) classifies the four damage mechanisms in 3 groups:

- Mode I: Self triggered ejection of fractured rock (strain burst or buckling with seismic source inside the failure mechanism)
- Mode II: Ejection of part of a fractured, broken or jointed rock mass (driven by inertia or stress waves)
- Mode III: Displacement of broken rock with gravity as dominant driving force component (seismically triggered falls of ground or enhanced gravity condition)

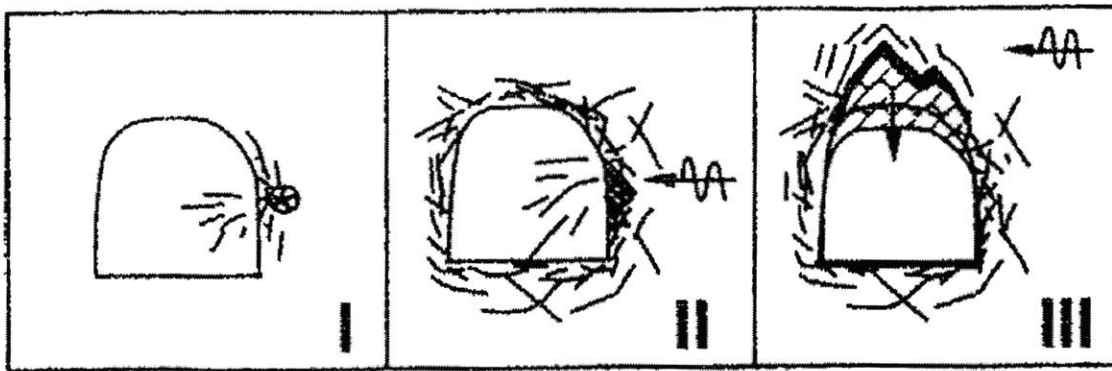


Figure 4.29: Categories of Failure from a Design Perspective (Ortlepp, 1992)

Ortlepp (1992) points out retaining and holding as support functions as do McCreath & Kaiser (1992) (Chapter 4.2). Ortlepp's experience in South African mines, where failure mode II and III are very common explains his emphasis of the holding element's ability to yield as the essential characteristic of a rock burst containing support.

4.6.2 Conceptual Design Considerations

4.6.2.1 Engineering Design Approach

Unfortunately a conventional design approach does not work, because there is insufficient knowledge to quantify the forces induced by rock burst failure mechanisms with sufficient accuracy to determine the capacity required to resist these forces. Hence empirical approaches must be used, but the fundamental components of engineering design must be included unconditionally. Even when ongoing research has developed suitable design charts a proper design will consist of the following elements:

- Identification of failure mechanism (Ortlepp, 1992)
- Elimination of potential failure processes by removing their cause. Or by preventing the identified failure mechanisms
- Identification where such failures are anticipated and whether they can be triggered. Most investigations start after an event has already occurred.
- Selection of a support system that can survive the impact. In order to allow an economical support surviving should be defined as maintaining functionality, even when the area has to be reconditioned.
- Identification of the maximum practical support limit (MPSL) beyond which elimination of the cause for damage constitutes the only viable solution (Kaiser, 1993).

Figure 4.30 illustrates Kaiser's (1993) suggestion of a conceptual design chart for mode II or mode III. The input parameters used are distance to the source mechanism and the event

magnitude. With those one can determine the most suitable out of three different types of supports. It is obvious, that the conceptual design chart (Figure 4.30) was directly derived from the stress field change at the target. Figure 4.31 shows the empirical relationship for the stress field change at Creighton mine.

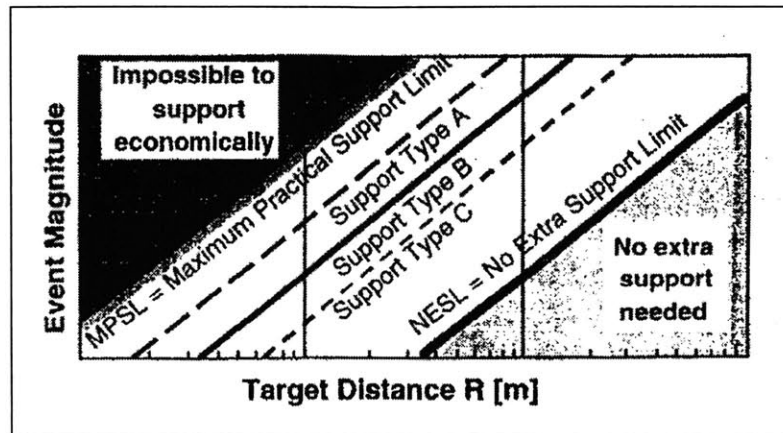


Figure 4.30: Proposed Conceptual Design Chart (Kaiser, 1993)

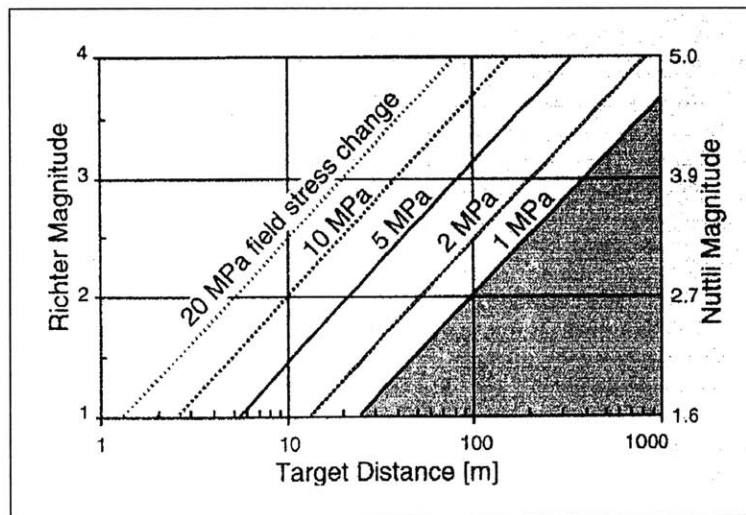


Figure 4.31: Empirical Relationship of Stress Field Change (Kaiser, 1993)

4.6.2.2 Factors to Consider for Mode II and III Support Design

Figure 4.32 schematically illustrates a seismic design sequence for mode II and III.

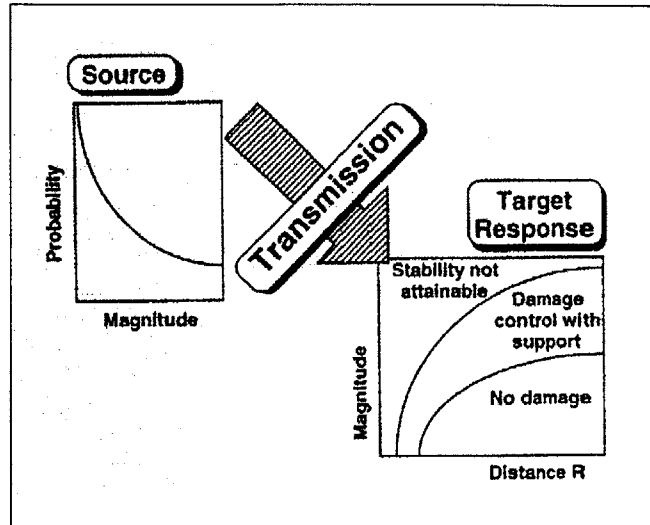


Figure 4.32: Seismic Design Sequence (Kaiser, 1993)

Although it is important for mode II and mode III to understand the source, the wave transmission and the response of the target properly, discussion will be limited, since it is mode I that is most likely to occur in civil engineering tunnels. For a matter of completeness it is pointed out that Jesenak (1993) carried out research on shielding the target from energy transmission. Also, mines should develop a “design seismic event” with the help of microseismic monitoring. Development of a statistical model and a rock burst risk index is planned by the mining industry to relate risk of occurrence to other mining parameters.

4.6.3 Design Examples for Mode I and II

As can be seen from the different damage mechanisms in Chapter 2 and the differences observed in support damage (Chapter 4.1), the loading imposed on rock support during a rock burst can be significantly different in magnitude and form from static instability. Below, support considerations for the two extreme loading conditions; the strain burst and ejection damage mechanisms are discussed. Hoek's Matrix (Figure 4.33) can be used for mode identification and determination of rock burst potential.

		<div> <div>More Jointing</div> <div>→</div> </div>		
		Massive	Moderately Jointed	Heavily Jointed
<div> <div>More Stress, More Fractures</div> <div>↓</div> <div>Low Stress</div> <div>No fractures</div> </div>		Massive rock with few joints • No burst potential	Discontinuous jointing No burst potential	Mode III Many joints with low stress favoring key block failures Potential of falls of ground
	<div> <div>More Stress, More Fractures</div> <div>↓</div> <div>High Stress</div> <div>No fractures</div> </div>	Mode I Massive but fractured rock near opening with few joints Strain burst potential	Mode I or II Discontinuous jointing but fractured rock near opening Buckling and shear burst potential	Mode III Many joints and fractures with potential for stress driven block failures Potential for falls of ground

Figure 4.33: Burst Potential & Failure Mode (Hoek, 1992 modified)

4.6.3.1 Design Considerations for Strain Burst Conditions

Strain bursting is a low energy surface phenomenon with the tangential stress around the cavity as single most important determinant of damage. A design for low capacity surface treatment should be adequate. Considering the bursting fragment as a column and determining the critical buckling stress gives qualitative prognostication. The critical buckling stress is given by:

$$\sigma_{cr} = \frac{\pi^2 Et^2}{3L^2} \quad \text{Equation 4.1}$$

where L is the column length, t the column thickness and E is Young's modulus.

One can see, that the thicker and shorter a fragment the higher the critical buckling stress σ_{cr} . From this simple relationship one can deduce two important influences of low capacity surface treatment:

- The column length is decreased by decreasing the spacing between rock bolts. To minimize exposed rock span it is advantageous to use closely spaced low capacity bolts with large face plates.
- The thickness of the fragment can be increased by applying shotcrete. Fiber reinforced shotcrete will create a more flexible element. It's application will therefore be advantageous.

In practice e.g. in the Norwegian tunnels the application of shotcrete, closely spaced rock bolts and large face plates has been proved to be effective in resisting the effects of strain bursts (Broch & Sorheim, 1984).

4.6.3.2 Design Considerations for Ejection Conditions

The energy makes it possible to design a support that can withstand rock ejection. The ejection velocity is probably the single most important determinant of the damage severity. Back calculations from ejection damage mechanism rock bursts with Newton's laws indicate that velocities of ejected blocks could be in the order of 8 m/s. Ortlepp (1992) confirmed this in simulated rock burst tests. The sizes of the ejected blocks depend on local jointing and fracturing around the excavation. Rock mass classification systems that include an implicit measurement of block size (e.g. Priest & Hudson, 1981) can provide an estimate of the size of the ejected blocks. It should be pointed out, that there is no simple relationship between the magnitude of the event that triggered the ejection and the damage. Hence peak particle velocity is not very useful in design for an ejection event.

As an example let us consider a single ejected bolt with a mass of 2 tons and a velocity of 8 m/s. The kinetic energy of the block would be $W_{\text{kin}} = \frac{1}{2} mv^2 = \frac{1}{2} * 2 * 8^2 = 64 \text{ kJ}$.

Figure 4.34 illustrates the failure for a yielding 22 mm bolt (i) and a rigid fully grouted 22 mm bolt (ii). In both cases the same block is ejected with 8 m/s. In the rigid case (i) the bolts fails and in for the yielding bolt halts after significant displacement.

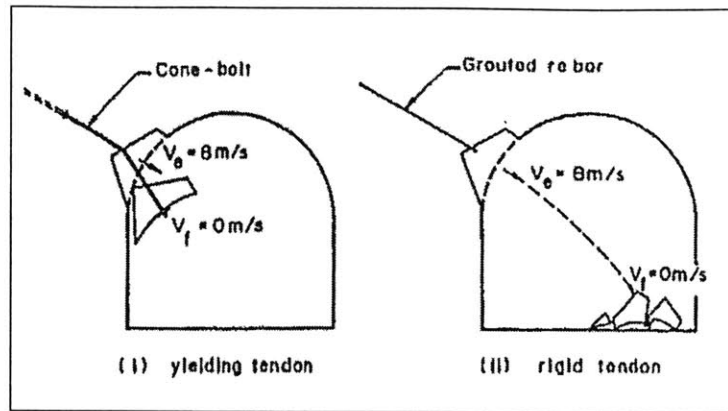


Figure 4.34: Energy Considerations for Yielding and Rigid Support (Ortlepp, 1992)

The load displacement curves for both bolts in Figure 4.35: explain why. The area under the load displacement curve represents the transformation of kinetic energy into deformation energy. In case of the yielding support element (i) such as a cone bolt (Jager et al.; 1990) the total energy of 64 kJ is consumed at a total displacement of 34 cm before the bolt fails. Therefore in case of a rock burst, the yielding bolt restrains the block from falling down. In the case of the stiff fully grouted bolt failure occurs after a displacement of 30 mm. The energy dissipated at this point is $0.03 \text{ m} * 160 \text{ kN} = 4.8 \text{ kJ}$, represented approximately by the rectangular abcd.

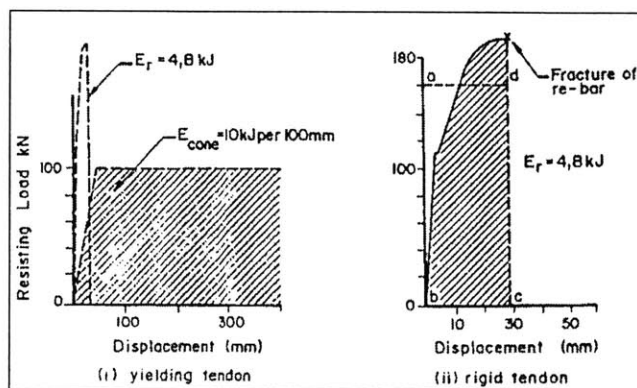


Figure 4.35: Energy Absorbed by Bolts (Ortlepp & Stacey, 1994)

$$\begin{aligned}
 v_{failure} &= \sqrt{\frac{2E_{kin,failure}}{m}} = \sqrt{\frac{2(E_{kin} - E_{absorb})}{m}} \\
 &= \sqrt{\frac{2(64 - 4.8)}{2}} = 7.7 \text{ m/s}
 \end{aligned}
 \tag{Equation 4.2}$$

That is about 8% of the kinetic energy and slows the block down to 7.7 m/s (Equation 4.2). This illustrates, that brittle conventional support alone is completely inadequate to resist a ejection rock burst loading.

From the example above and the energy based design rationale one can devise a more general design criterion for stability. It demands that the support has to be able to dissipate more energy than the kinetic energy of the ejected rock mass. Thus the energy resistance of the support has to be higher than the energy impact:


$$E_{R,support} \geq E_{kin} = \frac{1}{2}mv^2 \tag{Equation 4.3}$$

In a simple model where rock mass is suitable contained between the tendons, each tendon restrains a rock mass of a defined area with the thickness t. For the load of the tendon this simplistic model represents the worst case possible and is a justifiable conservative approach. In order to carry the load the tendons have to be able to dissipate more energy than the kinetic energy of the ejected rock mass per area.

$$\frac{E_{R,support}}{A} \geq \frac{E_{kin}}{A} = \frac{\frac{1}{2}mv^2}{A} = \frac{\frac{1}{2}\rho Vv^2}{A} = \frac{\frac{1}{2}\rho Atv^2}{A} = \frac{1}{2}\rho tv^2 \tag{Equation 4.4}$$

Only one design load parameter, the kinetic energy per square meter E_{kin}/A , is needed to carry out the design. As Equation 4.4 shows one needs three parameters to obtain E_{kin}/A .

The uncertainty of their exact determination varies significantly:

- Unit weight ρ
 - Thickness of the ejected rock slabs t
 - Ejection velocity v
- 
- Uncertainty increases

For tunnels with existing rock burst experience all parameters are known. The thickness can be measured and the ejection velocity can be obtained from back calculations.

For $\rho = 2.7 \text{ t/m}^3$ Figure 4.36 illustrates that the ejection velocity v , which is the most difficult to obtain accurately correctly, has the highest impact on the design load parameter. Figure 4.36 shows the kinetic energy of the rock mass as a function of the ejection velocity for various rock mass thickness t and Jager's (1988) proposal of a minimum requirement of 25 kJ/m^2 resistance.

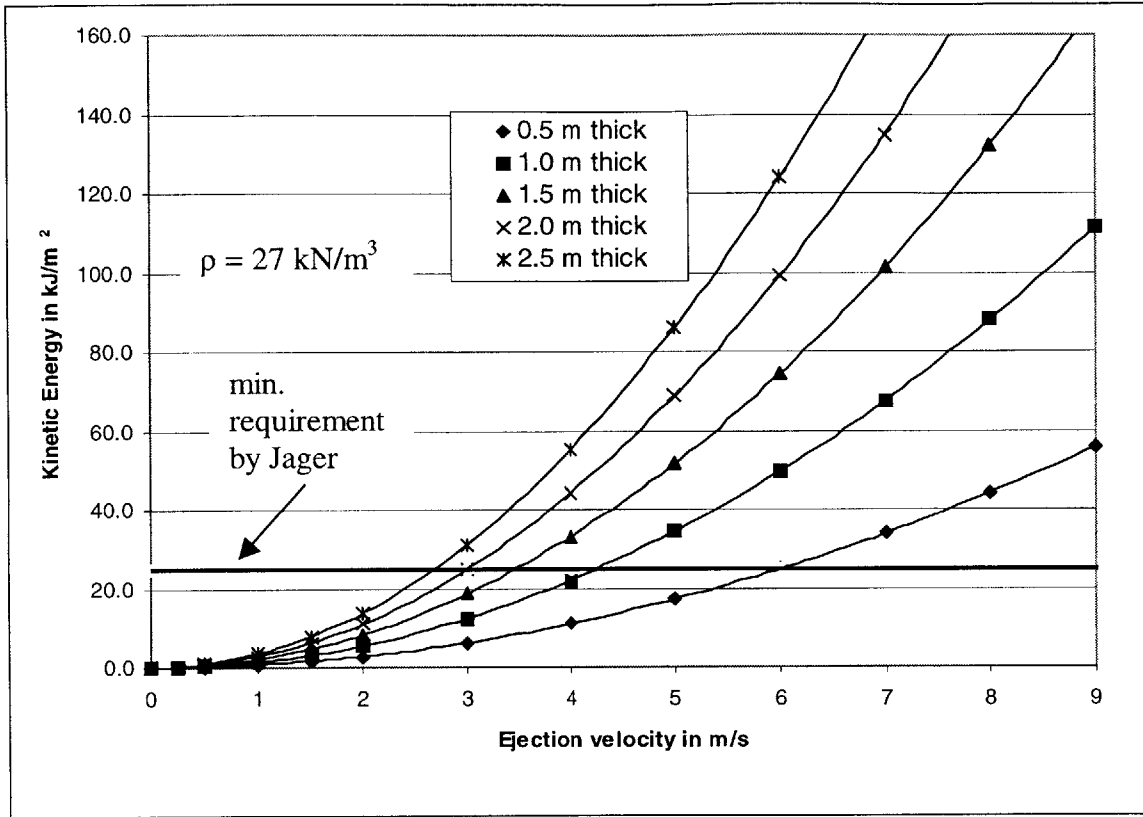


Figure 4.36: Kinetic Energy as a Function of Ejection Velocity and Thickness

The resistance parameters can be acquired with much higher precision. Only properties of the tendon and the tendon spacing need to be known to determine F/A , the force needed per m^2 to deform the tendons plastically. The energy dissipated by displacement results as

$$\frac{E_{R,\text{support}}}{A} = \frac{\int F(s)ds}{A} \text{ and with } F(s) \text{ as constant yielding force } F_y \text{ to } \frac{E_{R,\text{support}}}{A} = \frac{F_y * s}{A}.$$

The chamber of Mines of South Africa gives values for the unit resistivity F_y/A of the support. Knowing this and the ejection energy, the only additional criterion is that the support must yield. As design criterion of an “allowable” displacement could be

established. This might be necessary in order to provide functionality of the opening or if other reasons prohibit large displacements in parts of the tunnel. The first step is to determine the allowable deformation d_a . Knowing the kinetic energy per unit area and the force of the bolt (F) one can establish an equation to determine the displacement (Equation 4.5)

$$d_a > d = \frac{E_{kin}}{A} \frac{1}{F * n} \quad \text{Equation 4.5}$$

And solve for the number of bolts needed to be within the allowable displacement:

$$n > \frac{1}{d_a} \frac{E_{kin}}{A * F} \quad \text{Equation 4.6}$$

In the example used above and assuming one rock bolt per m^2 , the displacements are listed for three types of bolts:

- | | | |
|--|----------|--------|
| • Type SS 39 Split Set bolt stabilizer | 50 kJ/m | 1.28 m |
| • EXL Swellex dowel | 110 kJ/m | 0.58 m |
| • Cone Bolt | 190 kJ/m | 0.34 m |

For both the stability and deformability criterion the designer can reduce the spacing or use bolts with higher resistance, in order to fulfill the necessary conditions. One can see that it makes sense using more expensive cone bolts that dissipate four times as much energy as split set bolts, because less bolts are needed and the amount drilling gets reduced significantly. The objective should be to reduce the number of needed bolts in order to have economic support. For the maximum distance between the bolts an additional local design criterion for the local stability problem, like the one established for strain bursts earlier, has to be established.

4.6.4 Summary

Control of rock mass bulking during rock bursts is a key issue for tunneling. A proper support system must be able to hold and retain the broken rock but, more importantly, it must control and minimize the bulking process. This demands a well-engineered support system that reinforces the rock, can yield significantly, and connects the holding elements to a strong but flexible retaining element (Kaiser & Tannant, 2000).

Thus the implications for support design are: First, support for strain-bursting conditions can be achieved by increasing the density of low-capacity support like closely spaced bolts, faceplates and shotcrete. Second, that it is neither practical nor economically feasible to prevent severe rock burst damage from the ejection type damage mechanism by only increasing the strength of the existing tunnel support. Adding yielding support is necessary. As a result, tunnels designed for the loading case “block ejection” must be designed using an energy approach, which has to be constructible using presently available components. Figure 4.37 shows which McCreath & Kaiser’s (1992) support suggestions for increasing levels of rock bursts. The diagram was established for mining purposes, but can be used as guideline for civil engineering tunnels if the inherent differences are kept in mind.

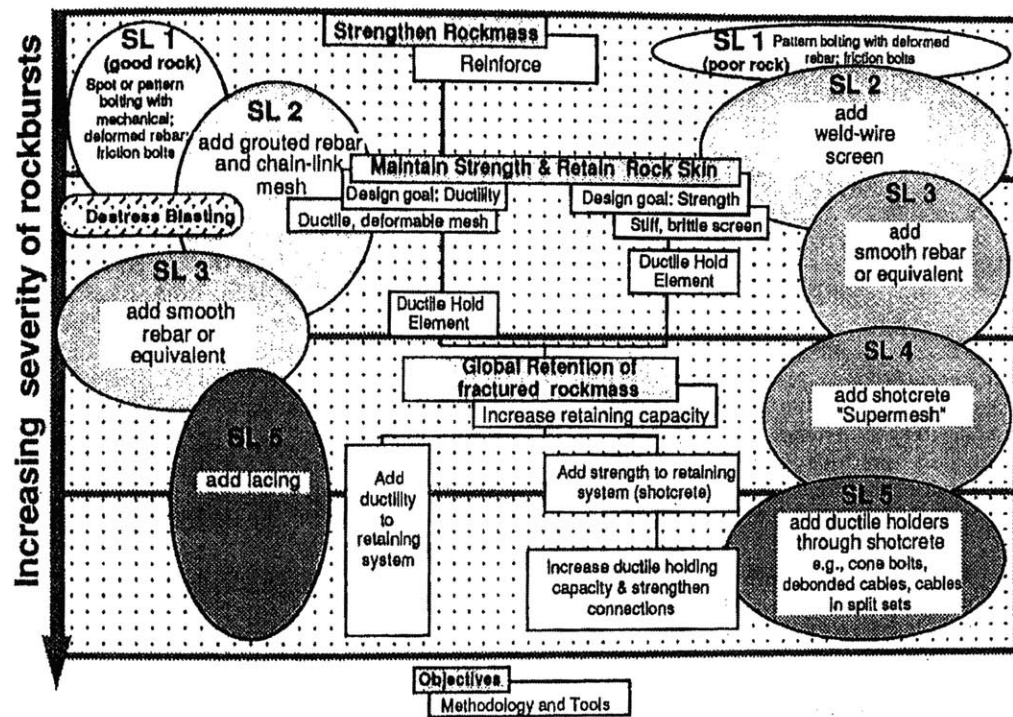


Figure 4.37: Guidelines for Support Levels in Burst Prone Ground (McCreath & Kaiser, 1992)

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Chapter 5 Prediction and Prevention of Rock Burst

5.1 Prediction

Seismic networks are installed in mines, where rock bursts are expected or have been experienced to make the identification of hazardous areas possible and aid the back analysis of rock burst events. The main objective is to determine parameters, which are most useful as indicators of increased rock burst hazard. Rock burst prediction research focuses on seismic data. Mining compared to tunneling causes significant seismic activity. Today, even with careful analysis of seismicity data, a reliable and timely prediction seems to be only a remote possibility for mines. It can be concluded that remote timely and spatial rock burst prediction for tunneling will not be an available option anytime soon.

Until more reliable methods of prediction are available the “old fashioned” method of visual inspection has to be used to identify the main indicators of unexpected rock bursts. The severity of rock bursts varies unexpectedly for two main reasons. One related to the varying conditions of the rock mass. The other is failure of inadequate support systems. Given the uncertainty and risk involved, frequent inspection of working place should be conducted by personnel which is able to identify changes in the rock mass condition, and recommend and implement appropriate changes in the support system.

5.2 Prevention

The ability to control the consequences of a rock burst with support measures is rather limited. Sometimes it is more economical to eliminate the triggering mechanism or to minimize the severity of rock bursts. Two of the most effective ways to prevent the triggering of strain bursts are modifications of the excavation shape and destress blasting. Ground support is used to minimize or control their severity.

5.2.1 Elimination of Triggering Mechanisms

To eliminate the trigger mechanism of type I rock bursts the stress level in the rock near the tunnel surface must be reduced or the rock must be reinforced. Several techniques can be used to achieve this.

5.2.1.1 Designing and Building a Tunnel to Minimize Stress

In elastic rock the optimal cross section for a tunnel often is elliptical. However this is often not practical. It is also not effective in cases where the stress around the tunnel exceeds the rock's strength. From a design perspective corners should be avoided to reduce local stress concentrations. However, practical experience has shown that corners, which act as stress raisers, are in fact beneficial because spalling occurs before significant levels of strain energy can be stored near the excavation surface (Kaiser & Tannant, 1999).

5.2.1.2 Reinforce the Rock Mass

The rock mass can be strengthened by reinforcement. Installation of fully grouted rebar, can increase its strength and therefore reduce the stress to strength ratio. Unfortunately the effect of rebar, in brittle moderately jointed rock is minor. The only damage mechanism that can be entirely prevented from occurring is low energy strain bursting. Shotcrete and a dense bolting pattern have worked successfully.

5.2.1.3 Energy Release Control

By encouraging incremental failure processes the energy release can be controlled. Energy release can be considerably reduced if a progressive, incremental failure process can be induced. This can be done e.g. by temporarily overstressing the rock with blasting. It should therefore be taken into consideration, that tunneling techniques using tunnel boring machines or smooth blasting are likely to create unfavorable conditions as far as rock bursts are concerned.

5.2.1.4 Use of Destress Blasting

Destress blasting was first used in mining in the early 1950s and is widely used today. Despite the apparent success of the destress blasting technique it is implemented without dedicated design analysis tools exist up to date. It is used to reduce the potential occurrence of a rock burst by creating or broaden a fracture zone around the cavity. The objective is to relieve high stresses present in the rock around the opening. Destress blasting involves regularly setting off carefully tailored blasts in the fractured rock immediately ahead the

tunneling face, so as to facilitate slip on pre-existing fractures, in order not to allow the accumulation of high strain energy density in the rock mass. Destressing has been applied in mining in zones of high stress and brittle rock.

Figure 5.1 shows a typical destress blasting arrangement as suggested by O'Donnell (1992). Destress blast holes form a fan and reach out beyond the planned perimeter of the tunnel. Three types of destress blast holes are distinguished according to their location: face holes (F), corner holes (C) and wall holes (W). They are about twice as long as the blast holes for a regular round and only the portion that exceeds the regular round length is loaded with explosives. Wall holes are only used in areas where two parallel tunnels are closer than one tunnel diameter away from each other.

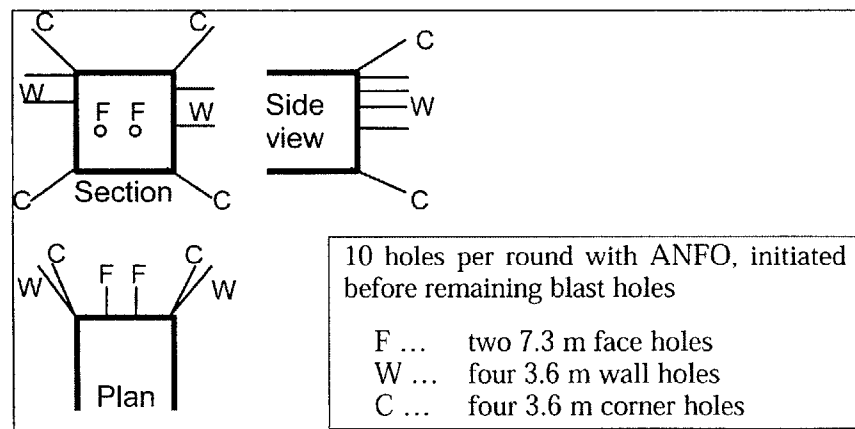


Figure 5.1: Destress Blasting Pattern (O'Donnell, 1992)

The distress blasting load is detonated with the explosives of the regular round. The blast damages and fractures the rock ahead of the new face and around the perimeter of the new round. It degrades the quality and strength of the rock to some extent and therefore reduces and redistributes stresses away from the tunnel deeper into more solid and better confined rock, where bursting is unlikely to occur. Distress blasting should be designed to maintain the tunnel profile, but fracture the rock sufficiently to reduce stresses. No excess damage to the rock should be incurred, so that additional support can be avoided.

Disadvantages of distress blasting however are the required fine-tuning of a satisfactory blasting arrangement and the delays. Distress blasting takes too much time for civil engineering tunneling project where delays are very costly. In addition it is only effective for a short period of time until the rock mass reacquires its original state of stress. Further drilling blast holes into highly stressed rock can result in additional challenges like blast hole spalling or steps in the blast holes (p. 104-105).

It has been observed (Stillborg, 1990), that drilling of holes for blasting and support has a similar effect on the rock, but not as pronounced as distress blasting. It has also been observed (Stillborg, 1990), that the distressing effect increases when pressure is applied to the boreholes during installation of Swellex rock bolts.

5.2.2 Minimization of Rock Burst Consequences

If the methods described in Chapter 5.2.1 fail, support (Chapter 4) is primarily used to reduce the consequences of rock bursts. The presence of support actually has no effect on the depth of failure for moderate to severe bursts (Kaiser & Tannant, 1999). Its primary function is to reduce bulking and to absorb energy. An effective support system must be able to hold back falling rock and control the deformation process (bulking). Different support components need to be combined, following the guidelines laid out in Chapter 4, to create an effective burst resistant support system.

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Chapter 6 Conclusions

This thesis described different mechanisms of rock bursts with a clear distinction between source and damage mechanisms. Most attention has been given to the damage mechanism strain bursting and ejection, which are the most likely in civil engineering tunnels. For strain bursting source and damage mechanisms coincide, and the event is of moderate violence. Depth of failure in strain bursting conditions is finite and determined by the static in situ-stress conditions. Rock ejection with ejection velocities of up to 10 m/s is imposed by seismic waves and can be of extreme violence and cause severe damage even to heavy conventional support.

In situ stress conditions have a dominant influence on the likeliness of a rock mass to be burst prone. Anisotropy as well as high stresses increase the likelihood of a rock to burst, but are not a necessity. Mechanical characteristics also affect the likeliness to burst. Strain bursting is more severe in hard, strong and brittle rock. However it can also occur in other rocks. The maximum strain energy per unit volume is a good measure to predict the violence a rock exhibits when bursting. Strain bursting is also more likely to occur in more massive rock types than in heavily jointed and fractured rock. Igneous and metamorphic rocks are more likely to be burst prone than sedimentary rocks. If geologic features like dykes and faults are encountered an increase in rock burst activity was recorded. It has been found, that strain bursts are encountered more frequently when using a TBM instead of drill and blast in otherwise similar conditions.

Support for strain bursting conditions needs prevent loosening of the rock mass between the tendons or hold loosened slabs in place. For rock ejection conditions a well-engineered support system that reinforces the rock, can yield significantly and connects the yielding holding elements to a strong but flexible retaining element is needed.

Increasing the density of the ordinary support used in non-bursting ground can make support suitable for strain bursting conditions. Shotcrete has been found to make the rock mass more ductile, thus reducing the rock mass' strain bursting potential. Against ejection it is necessary to use an energy approach for design rather than stress and strength considerations.

Therefore, yielding holding elements, which do not abruptly fail, but get pulled out of the rock mass, are desirable, due to their high energy dissipation. Good experience has been made with Swellex, split set bolts and cone bolts. In rock where severe ejection type bursts are expected, cone bolts should be considered. Yielding support systems can be designed to withstand rock bursts of major violence, but it is neither practical nor economical to design support for rock bursts of extreme violence.

Adequate prediction of rock burst is far from possible. Therefore further research should focus on prevention and reducing the impact with adequate support.